COPY FOR MR. J. ALLAN ROSS

HYDRO-ELECTRIC INQUIRY COMMISSION

ENGINEERING DATA

THE QUEENSTON-CHIPPAWA POWER DEVELOPMENT

CHAPTER "D"—POWER AVAILABLE

WALTER J. FRANCIS & COMPANY

CONSULTING ENGINEERS

Chapter D.

POWER AVAILABLE

Walter J. Francis.

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Chapter D.

POWER AVAILABLE

Walter J. Francis

General.

The determination of the power available from the Queenston-Chippawa Power Development involves, primarily, the determination of the elevation of the head waters, the carrying capacity of the Welland River, the carrying capacity of the Canal, the determination of the elevation of the tail water, and the efficiency of the main units. In addition to these principal features, many minor hydraulic losses have to be considered,- as, for example, the entrance head at the Intake, the losses due to bends and to changes of section in the River and in the Canal, the loss in entrance head at the screen house, the loss in the penstocks and in the Johnson valves, and in the draft tubes, as well as the losses in the turbines and in the electrical equipment.

The determination of available power for a given hydro-electric development generally includes amongst other factors watershed area, precipitation, evaporation and run-off, in order to arrive at the dependable flow, but none of the four enter into the problem in the usual way in connection with the Queenston-Chippawa Power Development. Lake Erie constitutes the immediate head water, and the discharge of the Great Lakes System down to and including Lake Erie, being the Niagara River, is the source of water supply for the Development. The

proportion of the discharge available for the Queenston-Chippawa Power Development is derived from the terms of an International Agreement. It is probable in our opinion that the quantity of water divertible from the Niagara River for hydro-electric power purposes may be increased in the near future by mutual consent of the United States and Canada. We think it probable in view of the advantages that would accrue that the elevation of the water surface at which the diversion may be made will be held more nearly constant, or, indeed, raised above the elevation now generally obtaining. The Development is also more advantageously situated than is usual in having its tailrace so closely adjacent to Lake Ontario. These features, briefly summed up as fixed and dependable elevations of head water and tail water, as well as definite flow, taken in conjunction, make the location of the Queenston-Chippawa Power Development highly advantageous from the viewpoint of hydraulics.

The various points are dealt with in the subsequent discussion.

All of the hydraulic characteristics have been considered by the engineers of the Hydro-Electric Power Commission, and their records are replete with valuable information in the form of data and calculations, many of which extend into the realms of research far beyond the bounds of the reference books on hydraulics. This information has been reviewed by us, the studies of the intake elevations, tail water elevations and canal losses having been more particularly followed by Lt.-Col. D. S. Ellis, M. A., during many months of the time he was a member of our staff and before he returned to resume his duties on the professoriate of Queen's University last October. He was eminently qualified by long experience as a hydraulician to pursue these studies,

and on relinquishing his place he was succeeded by other competent members of our staff who took up the subject.

The tests on the flow capacity of the Canal and the efficiency of the hydraulic equipment have been under the immediate direction of Mr. H. G. Acres and Mr. Thos. H. Hogg of the Hydraulic Department of the Hydro-Electric Power Commission, while the tests of the electrical equipment have been under the direction of Mr. E. T. J. Brandon and his staff, of the Electrical Department of the same Commission.

Taken as a whole, the results in the present volume under the general title of "Power Available" have been obtained after a practically continuous study during sixteen months, followed as quickly as the exigencies of the situation would permit.

The Location of the Development.

The location of the Queenston-Chippawa Power Development in relation to Lake Erie and Lake Ontario is clearly shown on the map included herewith as page D-4, while the succeeding map on page D-5 shows the relation of the Development diagrammatically in regard to the Niagara River. As these points have all been fully covered in previous Chapters further reference is unnecessary.

The Meteorological and Other Data.

The meteorological and other data with reference to the district are very

 Mr. J. Allan Ross.

and so relinquishing his place as was requested by other competent members of our staff who undertook the subject.

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Taken as a whole, the results in the present volume under the general title of "Power Available" have been obtained after a remarkably continuous study during sixteen months, followed as quickly as the examination of the situation would permit.

The Location of the Development.

The location of the Queenston-Chippawa Power Development in relation to Lake Erie and Lake Ontario is clearly shown on the map included herewith on page 3-4, while the accompanying map on page 3-5 shows the relation of the development diagrammatically in regard to the Niagara River. As these points have all been fully covered in previous Chapters further reference is unnecessary.

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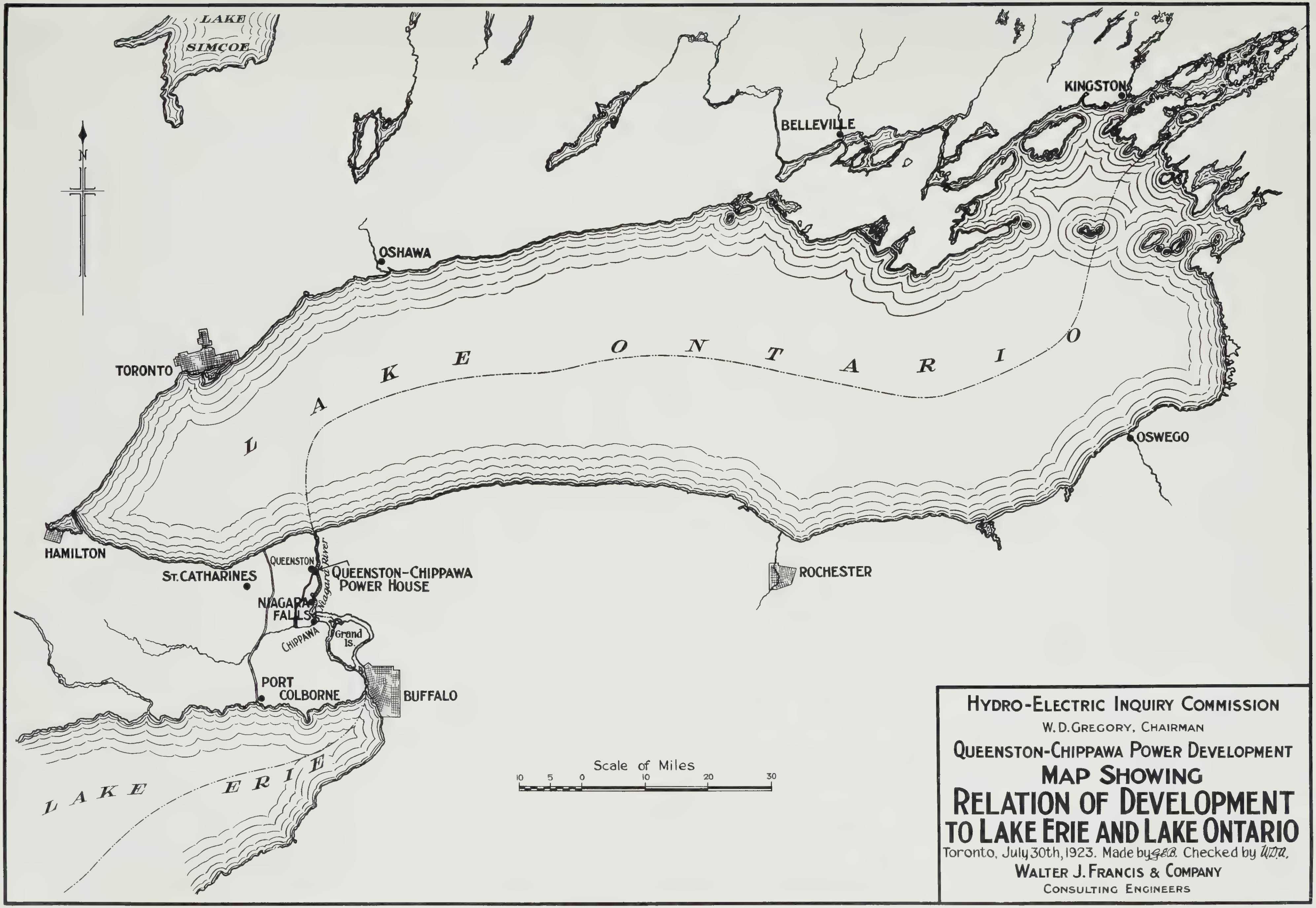
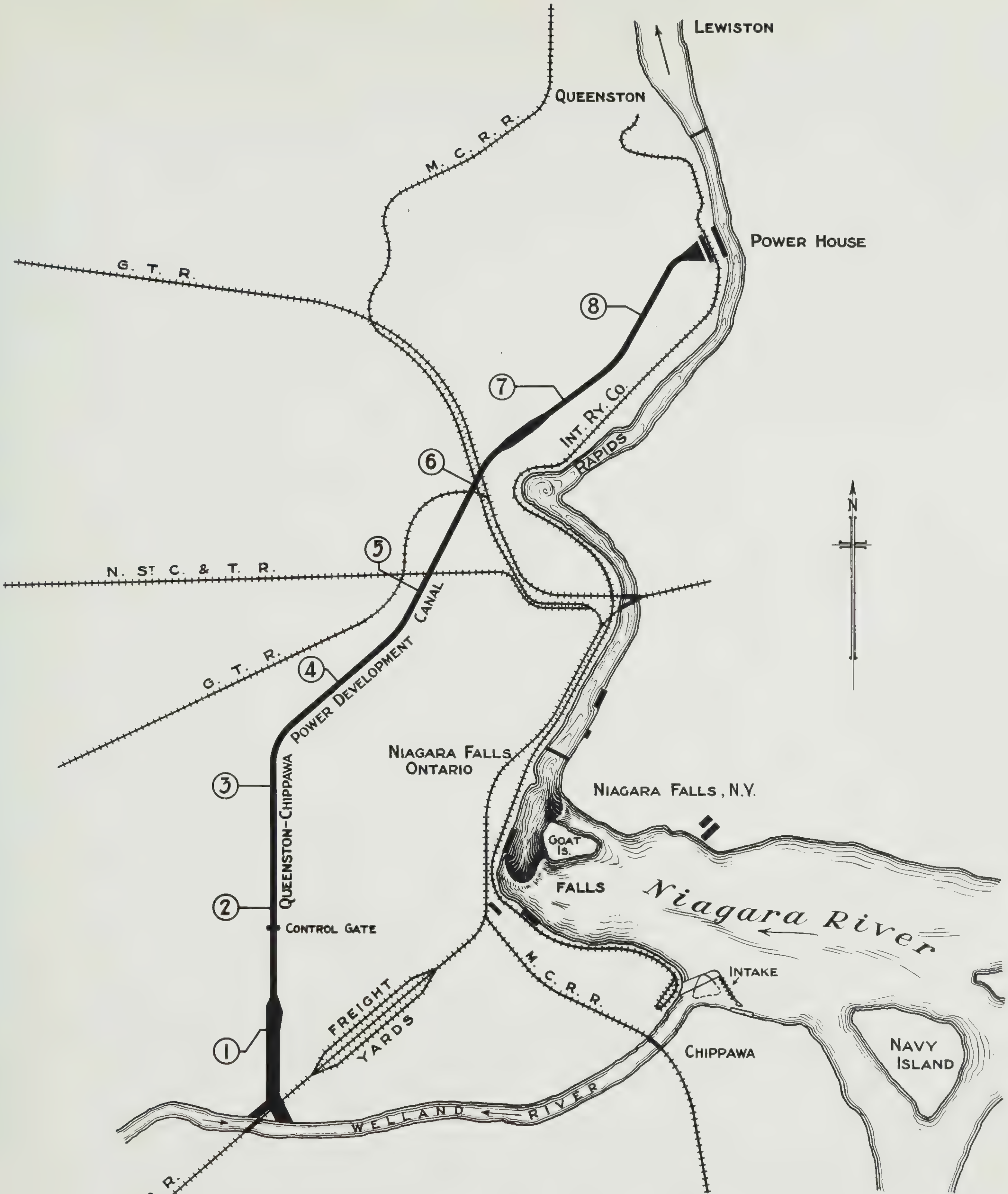

LAKE SIMCOE
N
OSHAWA
TORONTO
BELLEVILLE
KINGSTON
LAKE ONTARIO
OSWEGO
HAMILTON
QUEENSTON
St. CATHARINES
QUEENSTON-CHIPPAWA POWER HOUSE
Niagara River
NIAGARA FALLS
CHIPPAWA
Grand Is.
PORT COLBORNE
BUFFALO
ROCHESTER
LAKE ERIE
Scale of Miles
10 5 0 10 20 30
HYDRO-ELECTRIC INQUIRY COMMISSION
W. D. GREGORY, CHAIRMAN
QUEENSTON-CHIPPAWA POWER DEVELOPMENT
MAP SHOWING
RELATION OF DEVELOPMENT
TO LAKE ERIE AND LAKE ONTARIO
Toronto, July 30th, 1923. Made by G.E.B. Checked by W.D.M.
WALTER J. FRANCIS & COMPANY
CONSULTING ENGINEERS



NOTE:
Miles from junction with Welland River shown—1

Scale of Miles: 0, ½, 1, 2

HYDRO-ELECTRIC INQUIRY COMMISSION
W. D. GREGORY, CHAIRMAN
QUEENSTON-CHIPPAWA POWER DEVELOPMENT
GENERAL DIAGRAM OF DEVELOPMENT IN RELATION TO NIAGARA RIVER
Toronto, July 30th, 1923. Made by G.E.B. Checked by W.J.F.
WALTER J. FRANCIS & COMPANY
CONSULTING ENGINEERS

comprehensive and extend over a long period of time. In the preliminary studies the engineers of the Hydro-Electric Power Commission availed themselves of the records of the United States Lakes Survey, as well as the records of the various power companies operating in the Niagara district. As an illustration of the completeness of the records, two diagrams are enclosed herewith, being pages D-7 and D-8. These diagrams are reduced reproductions of the daily records kept by The Ontario Power Company from the time their plant was completed. The first of these records shows the year 1916, while the second one is for the year 1921. An examination of these two sheets will show the completeness of the data, that for 1921 including the ice conditions in the River Niagara which it was their custom to record. The wind and temperature records as well as the gauge readings make the information very complete. The records of the United States Lakes Survey are so well known that no further reference is necessary regarding them.

The Determination of the Elevation of the Head Waters.

The Chippawa - Grass Island Pool.

The intake for the Queenston-Chippawa Power Development is located at the confluence of the Welland River and the Niagara River in what is known as the Chippawa - Grass Island Pool, a broad stretch in the Niagara River lying between Navy Island and the rapids leading to Niagara Falls. The choice of the Chippawa - Grass Island Pool as a level for the intake of the Queenston-

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The Determination of the Elevation of the Head Waters.

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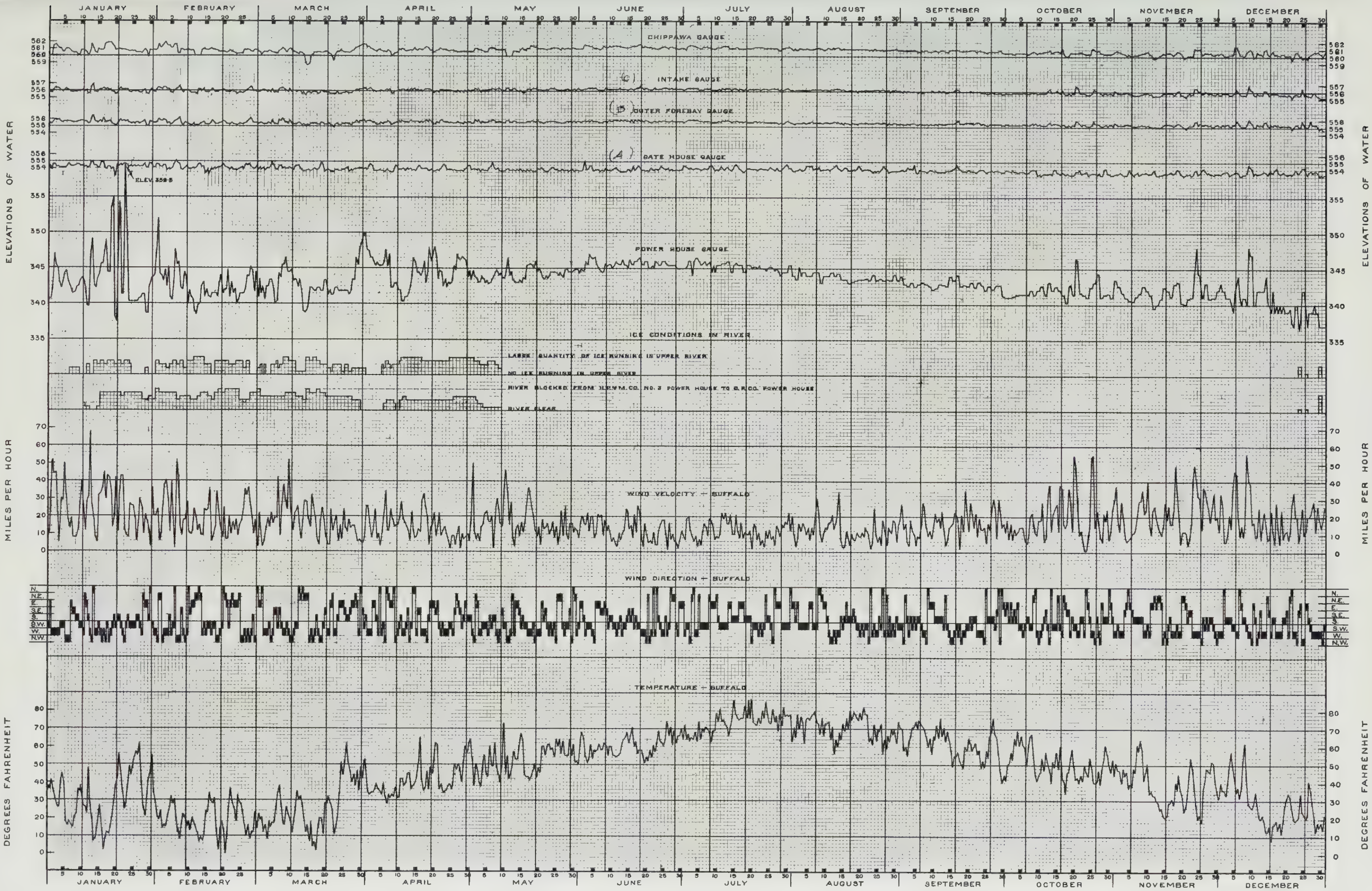


(D-7)

TEMPERATURE
Maximum:— 87°F. 8 P.M. July 20
Minimum:— 0°F. 8 A.M. Feb.21

WIND VELOCITY
Maximum:— 68 Miles per Hour 8 A.M. Jan 13

Records of temperatures, and of velocity and direction of wind, are obtained from published reports of U.S. Weather Bureau at Buffalo and are observations at 8 A.M. and 8 P.M. daily

LOCATION OF GAUGES	HIGH	DATE OF OCCURRENCE	LOW	DATE OF OCCURRENCE	MEAN FOR THE YEAR
CHIPPAWA	562·0	FEB.3	558·6	MAR.15	560·8
INTAKE	556·9	JAN.13, DEC.9	555·1	DEC.22, DEC.31	556·0
OUTER FOREBAY	556·5	JAN.13	554·7	DEC.22, DEC.31	555·5
GATE HOUSE	555·1	FEB.6, JUNE 2	552·8	JAN.20	553·9
POWER HOUSE	359·5	JAN.23	336·5	DEC.24	343·4

NOTE
569.864 O.P. Datum = 571.670 U.S.L.S. Datum.

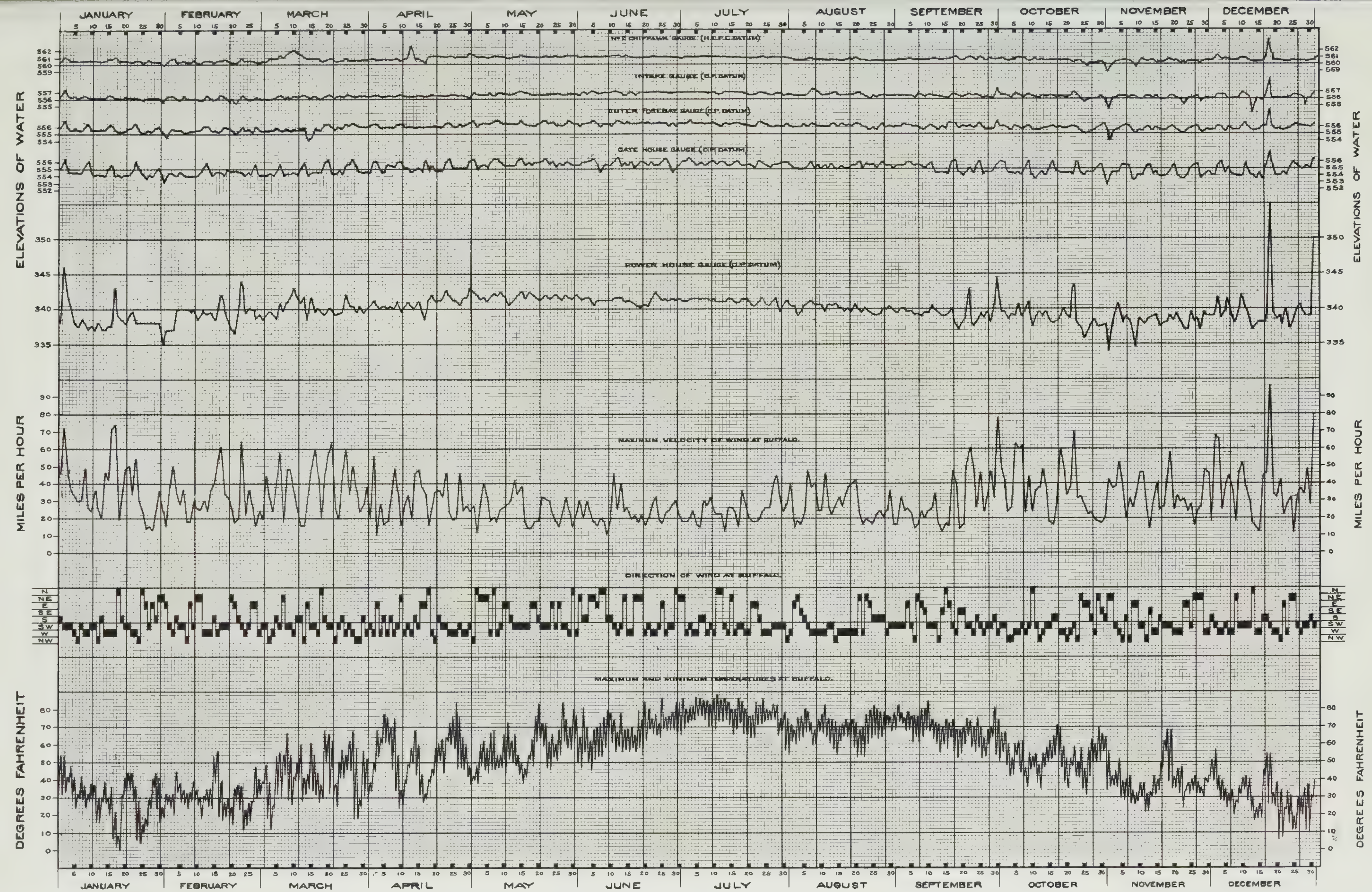
THE ONTARIO POWER COMPANY

DAILY RECORDS

NIAGARA RIVER GAUGES AND BUFFALO WEATHER CONDITIONS

1916

Compiled by W. J.



TEMPERATURE
MAXIMUM - JULY - 12 TH - 88°
MINIMUM - JAN - 19 TH - 0°

WIND VELOCITY
MAXIMUM - DEC. 18 TH - 96 MILES PER HOUR

RECORDS OF TEMPERATURES, OF VELOCITY AND DIRECTION OF WIND, ARE OBTAINED FROM PUBLISHED REPORTS BY U.S. WEATHER BUREAU AT BUFFALO, N.Y

LOCATION OF GAUGES	HIGH	DATE OF OCCURENCE	LOW	DATE OF OCCURANCE	MEAN FOR THE YEAR
CHIPPAWA	564.19	DEC. 18. TH	558.95	NOV. 1ST	
INTAKE	557.6	" " "	552.7	" " "	
OUTER FOREBAY	558.4	" " "	554.0	" 2 ND.	
GATE HOUSE	558.8	" " "	552.7	" 1 ST	
POWER HOUSE	355.1	" " "	334.2	" " "	

NOTE - 569.864 O.P. DATUM = 569.973 H.E.P. DATUM = 571.670 U.S.L.S. DATUM.

THE ONTARIO POWER COMPANY
DAILY RECORDS
NIAGARA RIVER GAUGES AND BUFFALO WEATHER CONDITIONS
- 1921 -

COMPILED BY. McLAUGHLIN

Chippawa Power Development was made after exhaustive studies had been made of the practicability of utilizing as much as possible of the total available head between Lake Erie and Lake Ontario. One of the reasons was that the Welland River could be utilized as part of the head works for several miles of its length, the natural flow being diverted into the artificial canal leading to the power house and the current being reversed from its natural direction in the portion of the Welland River between the canal junction and the mouth of the Welland River.

Owing to the comparatively large size of the Chippawa - Grass Island Pool and the way in which it discharges over the rapids below it, it may be considered as a basin from which the water flows over a broad-crested weir formed by the bed of the rapids leading to Niagara Falls.

The elevation of the water surface in this basin is of vital importance in considering the power available at the Queenston-Chippawa Power Development because the elevation of the water in the forebay at the screen house depends upon the elevation of the water in the Chippawa - Grass Island Pool, as well as upon the amount of water drawn through the canal.

A number of hydraulic problems are involved in the determination of the levels in the pool. It is necessary to define the possible limits between which the elevation of the water surface varies and also to fix the values which are most likely to occur therein. Besides the natural variation of stage, the effect of the diversion of a quantity of water out of its natural path over the falls into the power canal must also be considered, for the reason that such effects will be noticed not only in the pool itself, but also in the stage of

Lake Erie to a slight degree.

In studying the variations in stage of the Chippawa - Grass Island Pool the elevations were determined from gauge readings taken constantly since 1902 at or near the highway bridge over the Welland River at Chippawa Village, and from other gauge readings at Fort Day, the Buffalo break-water lighthouse, and at Cleveland. Continuous readings have also been taken from the gauge placed near the intake of the plant of The Ontario Power Company. The Fort Day gauge referred to is located at the head of the canal of the Niagara Falls Power Company near Niagara Falls, New York.

The Chippawa gauge was installed in 1902 as a staff gauge, and was read by officials of The Ontario Power Company until about 1915 when it was taken over by the Hydro-Electric Power Commission of Ontario. During this period there are a few gaps when readings were not taken, but these have been filled by interpolation from the series of readings taken at the intake of the plant of The Ontario Power Company. In November, 1919, a recording gauge was placed at Chippawa, and since that time its records have been used with the staff gauge readings as a check.

The readings of the Chippawa gauge appear to be reasonably consistent and, when combined with the records of the other gauges referred to, give fairly reliable evidence as to the stage of the water at Chippawa, which may be taken as the stage of the Niagara River at that point.

By plotting the monthly mean gauge height of the Chippawa gauge against the monthly mean of other gauges in this vicinity, the curve on which the points lie shows the relation existing between the stage at the two gauging stations

chosen. This procedure is necessary as a check upon the gauges, and also to transfer readings from one gauging station to another to supply deficiencies and to detect errors. Cross relations have been examined, comparing the Chippawa gauge with the Port Day gauge, the Chippawa gauge with the Buffalo gauge, and the Chippawa gauge with the Cleveland gauge.

Analysis of the Gauge Readings.

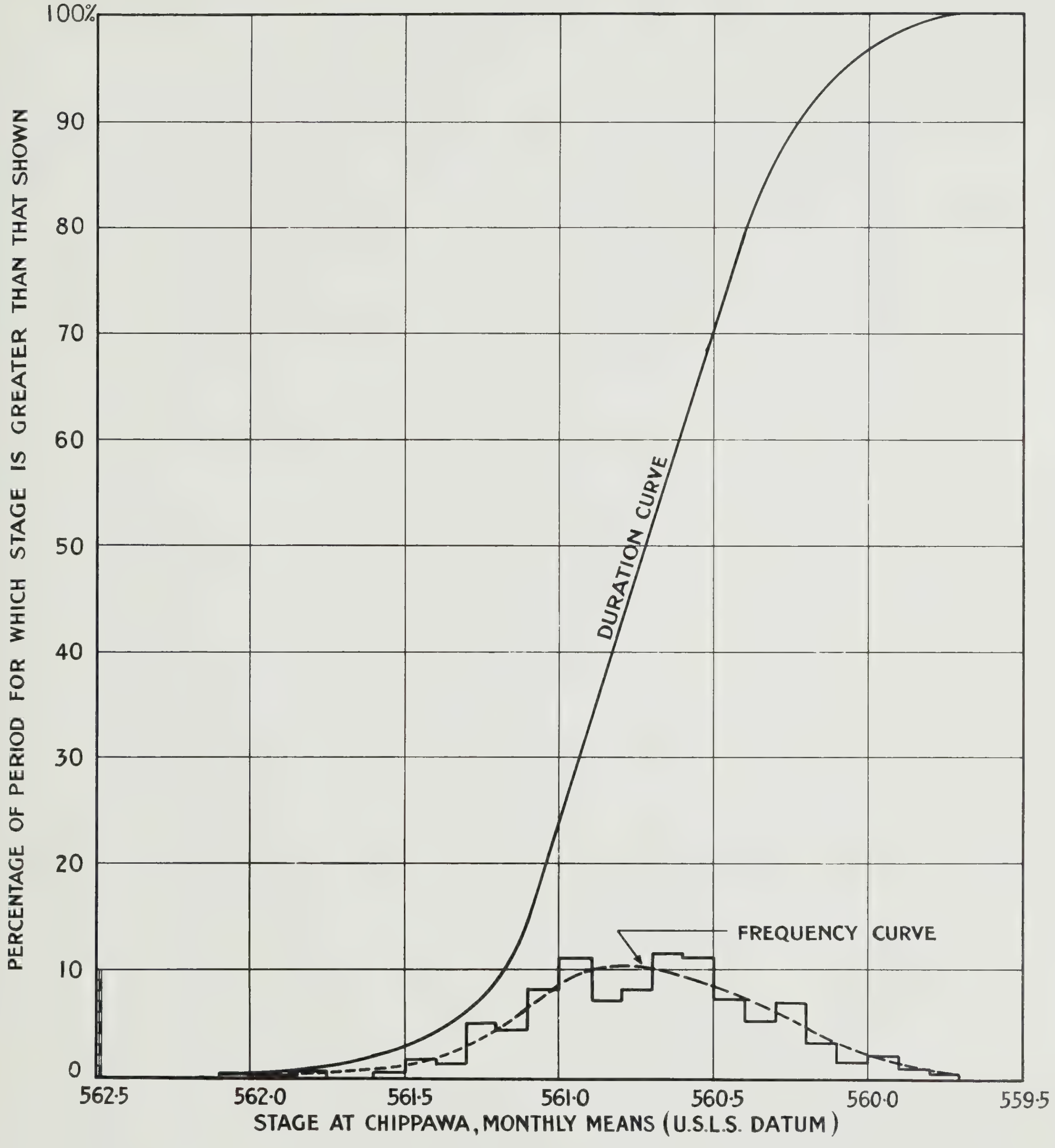
The power output obtainable at the Queenston-Chippawa Power Development depends upon the product of the volume of water used and the effective head through which it acts. The volume of water physically possible depends upon the stage at Chippawa and the hydraulic losses in the canal. The losses in turn are fixed with regard to maximum values by the construction of the canal invert and by the various elements of construction such as the intake, and the entrance to the penstocks. The net head available depends partly upon the stage at Chippawa, since upon it depends also the stage at the screen house, and partly upon the stage of the Niagara River at the point of discharge, which in turn is governed largely by the stage at Lake Ontario. It is therefore evident that the various critical values of the water stage in the Chippawa - Grass Island Pool must be closely studied. The most important of these values is that of the lowest stage at which the maximum amount of water can be discharged into the canal without allowing the water surface in the forebay to be below the minimum level for successful operation. This value for the water surface in the Chippawa - Grass Island Pool has been selected by the engineers of the Hydro-Electric Power Commission as Elevation 560.3, referring to the

datum of the Hydro-Electric Power Commission.

A curve has been developed based upon the montly mean stages at Chippawa showing the percentage of time throughout the period from 1902 to 1921, inclusive, when the stage in the Chippawa - Grass Island Pool has exceeded any given value. This curve, technically known as a "duration curve" is shown on the diagram on page D-13. On the same sheet a companion "frequency curve" is plotted for the same period, showing the frequency of each particular stage between the limits found in the gauge readings. From the duration curve it will be seen that the elevation which was selected as the minimum for operation occurs for about 84 per cent. of the time, and that Elevation 560.7 occurs for about 50 per cent. of the time. From the frequency curve the most frequent stage is seen to be at about Elevation 560.8.

The soundness of the decision of the engineers of the Hydro-Electric Power Commission in selecting Elevation 560.3 as the minimum operating stage at Chippawa must be examined in relation to the effect of power shortage, if any, which may result from it.

The determination of the monthly mean water elevations or stages does not give all the necessary information for a complete hydraulic study, for the reason that the water elevations present considerable daily variations from the monthly mean. As an illustration of these variations reference may be made to the records for 1916 and 1921, included as pages D-7 and D-8, which show the daily water elevations at Chippawa throughout the two years mentioned. An examination of this and of other similar diagrams shows clearly the method and the extent of the difference of the daily gauge heights from the mean for the



HYDRO-ELECTRIC INQUIRY COMMISSION

W. D. GREGORY, CHAIRMAN

QUEENSTON-CHIPPAWA POWER DEVELOPMENT

STUDY OF CHIPPAWA GAUGE

Toronto, July 30th, 1923. Made by S.R.W., Checked by W.D.A.

WALTER J. FRANCIS & COMPANY

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month. It will be seen from the diagram that during the summer months the difference referred to is not very great but that in the spring and fall violent fluctuations occur. As a rule the greatest fluctuation is that of increase in the daily stage above the mean, although the diagram for April, 1918, shows that sudden decreases in level also occur, in this case about 1.5 feet. It is apparent that these changes are almost entirely due to the influence of wind upon Niagara River and upon Lake Erie. The low stages in May, 1917, and in April, 1918, correspond to northerly winds, while the high water of November and December, 1917, as well as at other times, is found to correspond to westerly or south-westerly winds. A case is on record where the stage of Lake Erie was eleven feet higher at Buffalo than at Amherstburg during a very severe storm in which strong westerly winds prevailed.

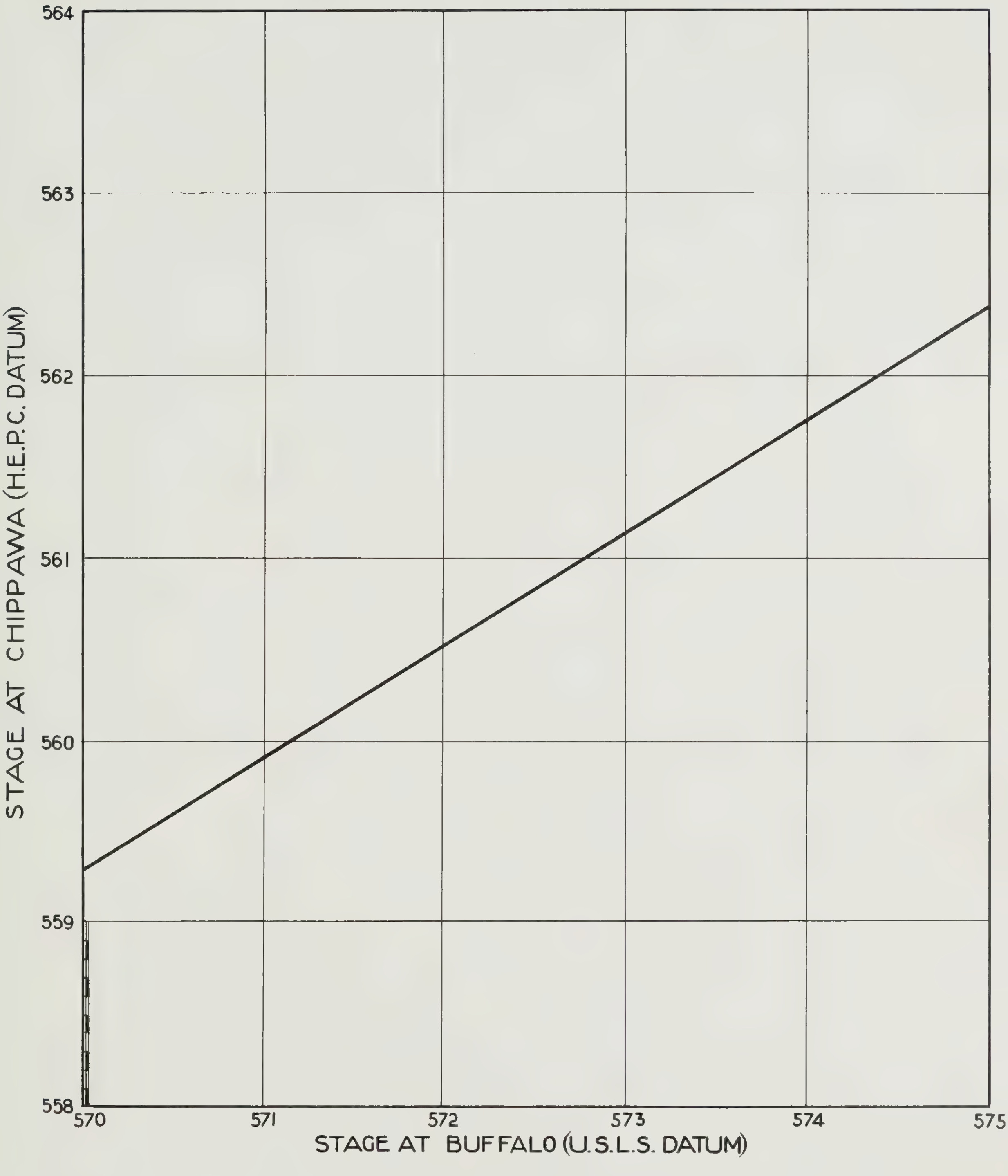
It must therefore be concluded that although the mean monthly stage is greater than Elevation 560.3 for 84 per cent. of the time, individual days will often show stages below that elevation due to temporary weather conditions. Such low water periods will occur during the time that strong northerly winds are most prevalent, namely, in the autumn and winter, which is coincident with the heaviest load demands on the plant.

<u>Co-relation of the Chippawa Gauge with Other Gauges.</u> As already mentioned, studies have been made of the relation of the Chippawa gauge to a number of other gauges nearby. As examples of these relations two diagrams have been prepared and included herewith as pages D-15 and D-16, the first of which shows the relation between the Chippawa gauge and that at the Buffalo break-water lighthouse, and the second the relation between the Chippawa gauge and that at

month. It will be seen from the diagram that during the summer months the difference referred to is not very great but that in the spring and fall violent fluctuations occur. As a rule the greatest fluctuation is that of increase in the daily range above the mean, although the diagram for April, [illegible], shows that extreme depression [illegible], in this case about [illegible] feet. It is apparent that these changes are almost entirely due to the influence of wind upon Niagara River and upon Lake Erie. The low stages in May, 1917, and in April, 1918, correspond to northerly winds, while the high water of November and December, 1917, as well as at other times, is found to correspond to westerly or southwesterly winds. A case is on record where the stage of Lake Erie was [illegible] higher than an [illegible] during a very severe storm in which strong westerly winds prevailed.

It must therefore be concluded that although the mean monthly stage is greater than [illegible] for 50 per cent. of the time, individual days will often show stages below that elevation due to temporary weather conditions. Such low water periods will occur during the time that strong northerly winds are most prevalent, namely, in the autumn and winter, which is coincident with the heaviest load demands on the plant.

Correlation of the Chippawa Gauge with Other Gauges. As already mentioned, studies have been made of the relation of the Chippawa gauge to a number of other gauges nearby. As examples of these relations two diagrams have been prepared and included herewith as Nos. [illegible] and [illegible], the first of which shows the relation between the Chippawa gauge and that at the Buffalo breakwater lighthouse, and the second the relation between the Chippawa gauge and that at



HYDRO-ELECTRIC INQUIRY COMMISSION

W. D. GREGORY, CHAIRMAN

QUEENSTON-CHIPPAWA POWER DEVELOPMENT

RELATION OF WATER STAGES AT BUFFALO AND CHIPPAWA

Toronto, July 30th, 1923. Made by , Checked by

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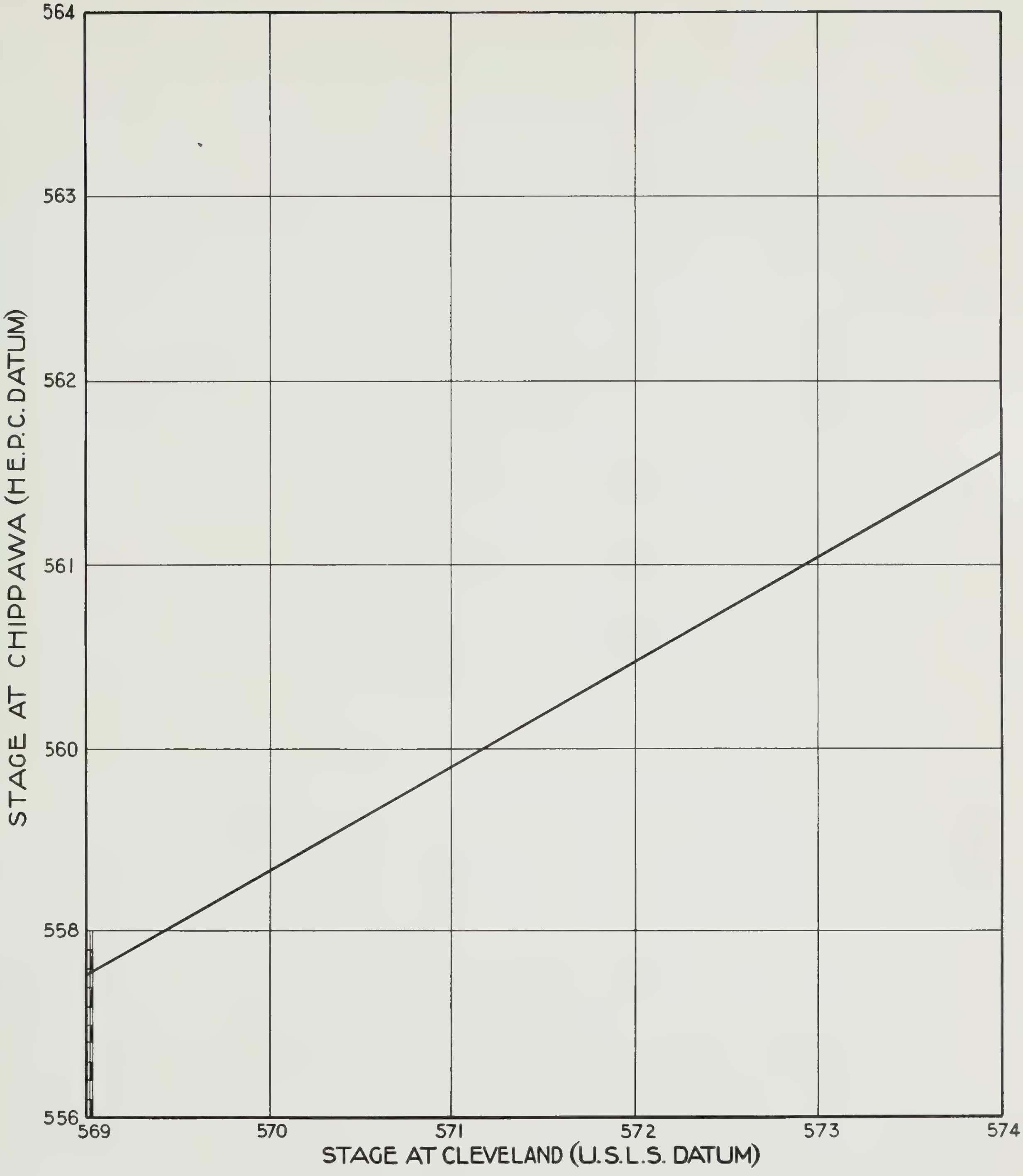
(D-16)



HYDRO-ELECTRIC INQUIRY COMMISSION

W. D. GREGORY, CHAIRMAN

QUEENSTON-CHIPPAWA POWER DEVELOPMENT

RELATION OF WATER STAGES AT CLEVELAND AND CHIPPAWA

Toronto, July 30th. 1923. Made by W.D. Checked by W.D.A.

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Cleveland.

It will be noted that the datum for the levels given for the Buffalo gauge is that of the United States Lakes Survey, while the Chippawa elevations are based on the datum of the Hydro-Electric Power Commission. From this diagram it will be seen that a change of one foot at Chippawa is equivalent to a change of about 1.7 feet at Buffalo.

In examining the past gauge records for Buffalo it was found that the monthly mean elevation was as low as Elevation 570.71, in November, 1895, and that in February, 1896, it fell to Elevation 570.69. By using the gauge relation diagram the corresponding levels at Chippawa are seen to be Elevation 559.85. In considering the levels of the Chippawa - Grass Island Pool the possibility of the recurrence of this extreme low water should therefore be borne in mind. Further, allowing for the fact that the daily level might reach 1.5 feet below the monthly mean, it is possible that a minimum level in the pool of Elevation 558.35 might occasionally be expected during a period when the level of the Great Lakes fell as low as it did in 1895 and 1896.

Other diagrams were used in the calculations and determinations of the levels, for example, a similar relation was plotted for the Chippawa - Fort Day gauge relations but is not included herein. From a study of all the data available it may be concluded that the levels in the Chippawa - Grass Island Pool may be determined within reasonably close limits, and that Elevation 560.3 is a proper figure.

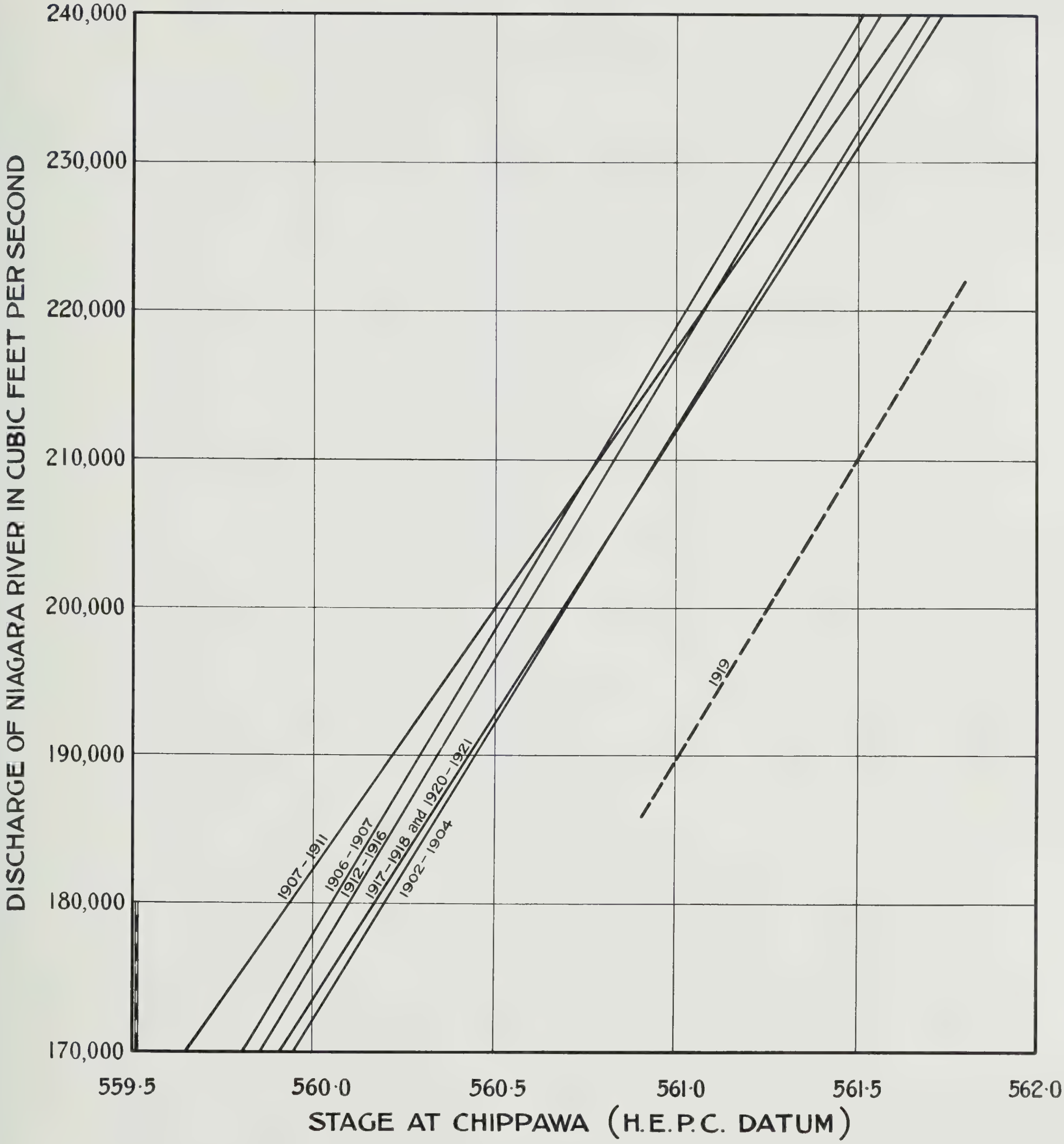
In 1920, Mr. R. S. Lea, M.E.I.C., made a special study of the level of the Chippawa - Grass Island Pool under the instructions of the Hydro-Electric

Power Commission of Ontario and submitted two reports of the conclusions reached. These are referred to in "Chapter C, Advisory Reports". Considering that the Chippawa gauge was set from the surface of the water at the highway bridge crossing, and that the water connection between this point and the Pool proper was by way of the northerly side of Hog Island when the gauge was set, we feel that an elevation several tenths of a foot higher than that chosen by Mr. Lea is justified from the single fact that the connection of the Welland River with the Niagara is now by way of the southerly side of Hog Island.

<u>Co-relation of the Chippawa Gauge with Niagara River Discharge.</u> In order to study the relation between the stage of the water elevation at Chippawa with the discharge of the Niagara River, curves have been prepared by plotting the monthly mean discharge of the river against the monthly mean gauge heights at Chippawa. The discharge of the Niagara River is computed from gauge readings on the United States Lakes Survey gauges at the head of the Niagara River and the gauge levels relating to the various discharges have been transferred to the Chippawa gauge readings. On page D-19 a series of six lines is shown representing the relation between the stage at the Chippawa gauge and the discharge of the Niagara River. It will be noted that there are five of the lines grouped. Each of the five lines represents the mean relation for the period indicated on the line, usually for several years. The separate line is for the year 1919, when the records show that something happened to raise the Chippawa stage about 0.5 feet and that during the following year it regained its ordinary regimen. It seems likely that this temporary change in elevation was due to depositing the dredged material from the Welland River in the

Survey Department of Ontario and submitted two reports of the conclusions reached. These are referred to in "Chapter 6, Advisory Reports". Considering that the Chippawa gauge was set from the surface of the water at the highway bridge crossing, and that the water connection between this point and the river proper was by way of the southerly side of Hog Island when the gauge was set, we find that an elevation several tenths of a foot higher than that shown by Mr. Ross is justified from the simple fact that the connection of the Welland River with the Niagara is only by way of the southerly side of Hog Island.

Correlation of the Chippawa Gauge with Niagara River Discharges. In order to study the relation between the stage of low water elevation at Chippawa with the discharge of the Niagara River, curves have been prepared by plotting the monthly mean discharge of the river against the monthly mean gauge heights at Chippawa. The discharge of the Niagara River is computed from gauge readings on the United States Lake Survey gauges at the head of the Niagara River and the gauge levels relating to the various discharges have been transferred to the Chippawa gauge readings. On page D-18 a series of six lines is shown representing the relation between the stage at the Chippawa gauge and the discharge of the Niagara River. It will be noted that there are five of the lines grouped. Each of the five lines represents the mean relation for the period indicated on the line, mostly for several years. The separate line is for the year 1918, when the records show that something happened to raise the Chippawa stage about 0.5 feet and that during the following year it reached its ordinary regimen. It seems likely that this temporary change in elevation was due to depositing the dredged material from the Welland River in the



HYDRO-ELECTRIC INQUIRY COMMISSION

W. D. GREGORY, CHAIRMAN

QUEENSTON-CHIPPAWA POWER DEVELOPMENT

STAGE-DISCHARGE CURVES AT CHIPPAWA, 1902 TO 1921

Toronto, July 30th, 1923. Made by G.E.B. Checked by W.D.A.

WALTER J. FRANCIS & COMPANY

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Niagara, and that this was washed away subsequently. The line for 1919 can probably be discarded as a permanent record.

The slopes of the curves show the effect of diverting the Niagara water out of its natural path into the Welland River for use in the Queenston-Chippawa Power Development. While the lines on the diagram on page D-19 are not all coincident they are all practically parallel, showing that, regardless of the cause of their non-coincidence, the relation of the increment of discharge to the increment of stage is constant. By scaling from the diagram itself it is found that an increase of one foot in the stage at Chippawa is equivalent to an increase in the discharge of the river of about 40,000 cubic feet per second. Hence, if 18,200 cubic feet per second be diverted from the river by the Chippawa Canal, a drop in the level of the pool of about 0.45 feet would result, because the reduced quantity of water passing over the Falls would require less head or energy to overcome the hydraulic resistance.

Remedial Works.

In passing it may be noted that a fall in level of 0.45 feet at Chippawa would produce a change in the level of Lake Erie on the order of 0.07 feet at low water stages, and less than this amount at higher stages of the river. It is probable that such a change in the natural regimen of Lake Erie would not be permissible and that remedial works might be required to compensate in level for the withdrawal of water for the power plants. Such remedial works would be designed to restore the level of the Chippawa - Grass Island Pool, or to hold it at or about its natural level during low stages of the river. In the

Copy for enclosure to Mr. J. Allen Ross.

Niagara, and that this was needed only subsequently. The line for 1919 can probably be discarded as a permanent record.

The slopes of the curves show the effect of diverting the Niagara water out of its natural path into the Welland River for use in the Queenston-Chippawa Power Development. While the lines on the diagram on page D-19 are not all coincident they are all practically parallel, showing that, regardless of the cause of their non-coincidence, the relation of the increment of discharge to the increment of stage is constant. By scaling from the diagram itself it is found that an increase of one foot in the stage at Chippawa is equivalent to an increase in the discharge of the river of about 40,000 cubic feet per second. Hence, if 10,000 cubic feet per second be diverted from the river by the Chippawa Canal, a drop in the level of the pool of about 0.25 foot would result, because the reduced quantity of water passing over the Falls would require less head or energy to overcome the hydraulic resistance.

Remedial Works.

In passing it may be noted that a fall in level of 0.25 feet at Chippawa would produce a change in the level of Lake Erie of the order of 0.07 feet at low water stages, and less than this amount at higher stages of the river. It is probable that such a change in the natural regimen of Lake Erie would not be permissible and that remedial works might be required to compensate in level for the withdrawal of water for the power plants. Such remedial works would be designed to restore the level of the Chippawa - Grass Island Pool, or to hold it at or about its natural level during low stages of the river. In the

prosecution of construction work for the hydro-electric plants above the Falls, rock spoil was deposited on the riverbottom in the region of the downstream part of the Chippawa - Grass Island Pool, and we are advised that this deposit had the immediate effect of stabilizing the elevation of the Pool to a slight extent. We are willing to accept this as a fact, and as a proof that remedial works may be so constructed as to procure the desiredeffect without detriment and at reasonable cost. The remedial works would also have to be designed to pass the high water flow, so as to prevent flooding. It will probably be necessary to consider the remedial works in connection with future negotiations between Canada and the United States in relation to further diversions of water from the Niagara River.

Hydraulic Studies of the Welland River and of the Canal.

Hydraulic Losses.

From the Intake at Chippawa to the Screen House various hydraulic losses of head occur, all tending to reduce the net head available for power production. It is therefore necessary to study the magnitude of these losses for the several elements of the Canal, and for various discharges of water through it, all related to the water stage at Chippawa. These losses vary approximately with the square of the velocity of flow, so that a drop in head of one foot at Chippawa is magnified in the resultant drop at the Screen House.

There are two general classes into which the losses may be grouped, one

being the loss due to friction on the wetted perimeter, and the other being that which is caused by changes of section and obstructions to general flow. The former class of losses is found in the stretches of the river and the canal, while the latter occurs at bridge piers, bends and transition sections, as well as at the intake and at the forebay. In this discussion the comparatively small losses due to curves in the Canal have not been taken into account, nor have those due to the restriction of cross-section due to bridge piers, because the losses are very uncertain and their values are small. The principal losses are considered in detail in the following discussion, the calculations having been made for two different rates of flow of water, namely, 17,000 cubic feet per second and 18,[illegible]00 cubic feet per second. These figures have been chosen as representing the probable upper limits of the continuous maximum flow capacity of the rock section of the Canal, which is the limiting feature of the present design.

<u>Intake</u>.

The present design of intake has provision for the future installation of submerged gathering tubes as described elsewhere in detail. It is not likely that these gathering tubes will be necessary until practically all of the main units will have been installed in the power house, thereby requiring comparatively high velocities of water through the Intake. While the plant is operating with less than eight units in service, the velocities near the Intake will be so low that it is probable that ice will not be drawn into the Canal, and therefore the gathering tubes need not be installed.

Assuming their necessity to have been demonstrated by service and the tubes installed, under adverse conditions of ice and with the full development, the sluiceways will be closed, and all the water will pass through the tubes from the Niagara River to the Welland River section. The passage of the water through the tubes will entail a loss of head due to its entrance into the tubes, a further loss of head due to friction in the tubes, and another loss because of the divergence in the discharge section of the tubes.

The Intake structure not having been completed when a former description was written, four photographs are included herewith as pages D-24, D-25 and D-26, to show the details. These pictures were taken just before the water was admitted, and the notes on the respective facing pages are fully explanatory.

It is estimated by Mr. R. D. Johnson, Consulting Engineer, in his report to the Hydro-Electric Power Commission under date of March 1st, 1920, that the loss of head in the type of intake proposed by him for adoption would be 0.53 feet for a flow of 15,000 cubic feet per second. Since the loss varies approximately as the square of the discharge, the loss for a discharge of 18,200 cubic feet per second would be about 0.78 feet.

From experiments on a working model made to scale, with a discharge of 1.45 cubic feet per second, the loss of head was found to be 0.77 inches. Similarly, with a discharge of 2.14 cubic feet per second the loss was found to be 1.63 inches, and for a discharge of 2.58 cubic feet per second the loss was 2.26 inches. Using these figures as a basis and deriving the value of "C" for the equation $q = CA\sqrt{2gh}$, the quantity being in cubic feet per second, the area being in square feet, and the head being in feet, and considering the

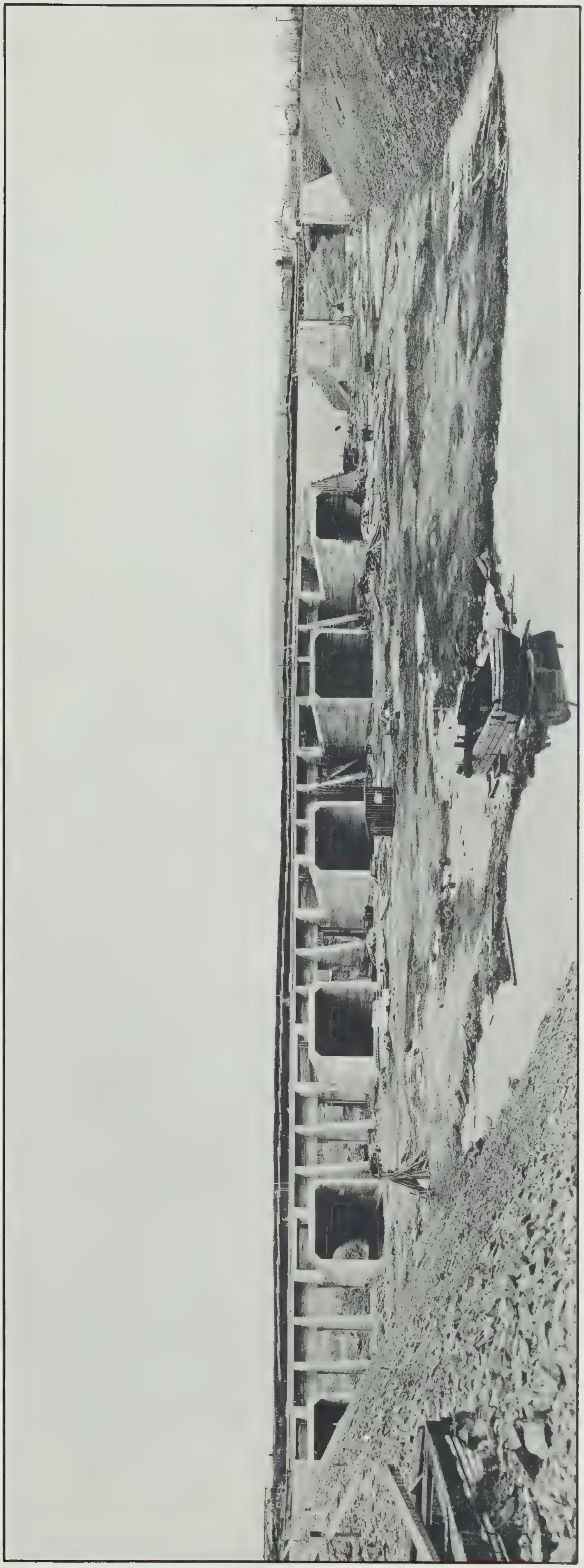
To face page D-24.

No. 1

Photograph showing

General View of finished Concrete Work
of
Downstream Side of Intake,

Taken December 4th, 1922.



To face page D-25.

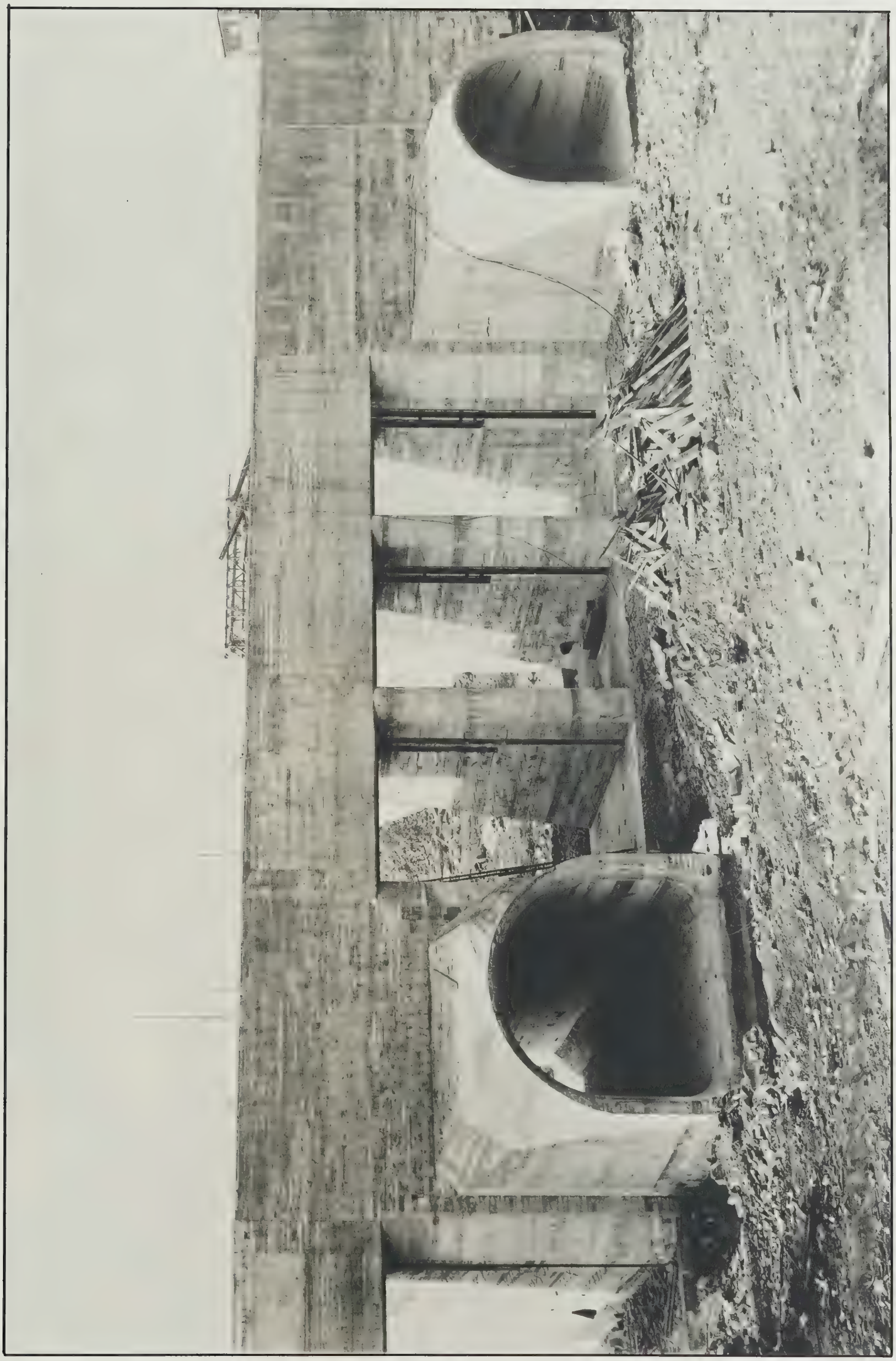
No. 2

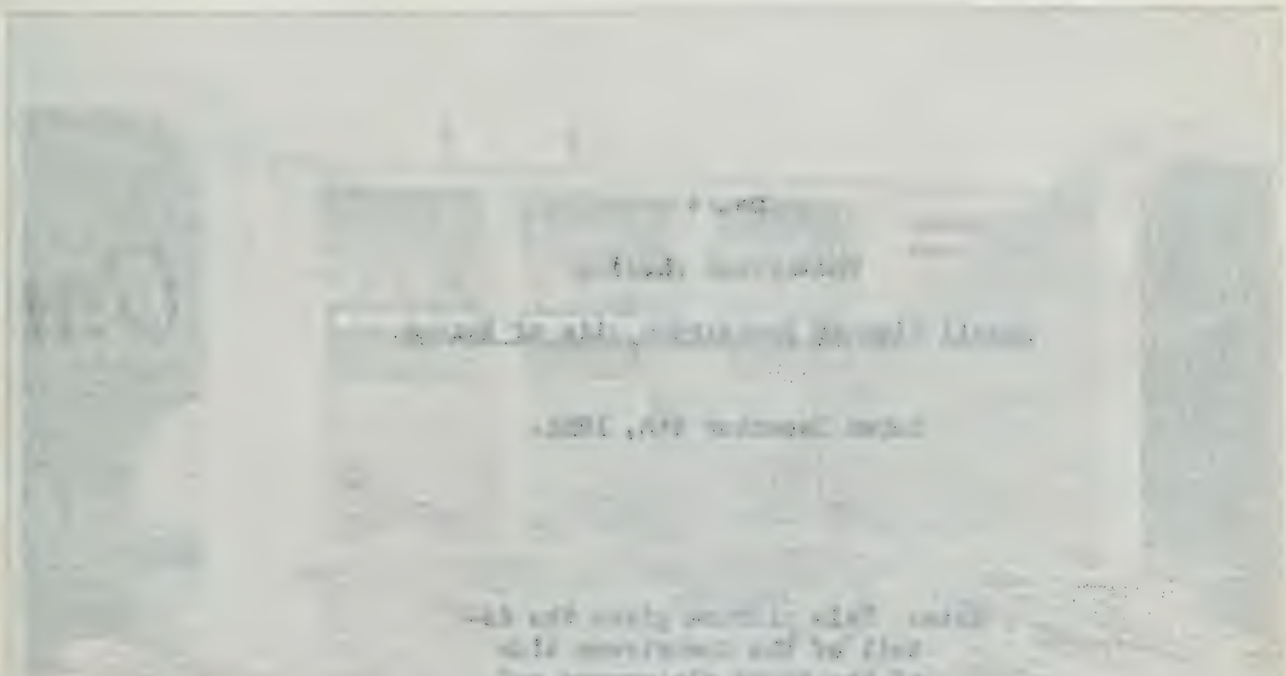
Photograph showing

Detail View of Upstream Side of Intake.

Taken December 4th, 1922.

Note: The upstream end of two of the submerged tubes, ready for future extension upstream, and the intervening sluiceways and ice booms may be seen as completed ready for the admission of water.





No. 3

Photograph showing

Detail View of Downstream Side of Intake.

Taken December 4th, 1922.

Note: This picture gives the detail of the downstream end of one of the submerged tubes as completed.

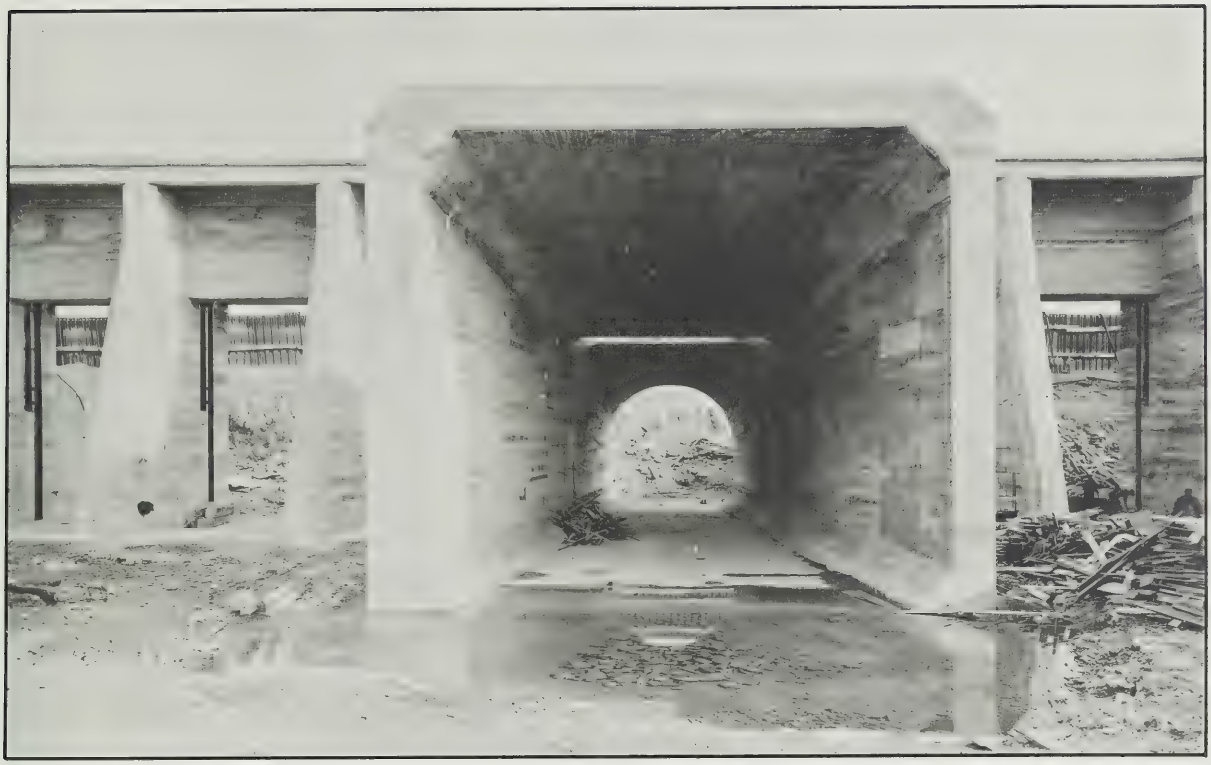
No. 4

Photograph showing

Detail View of Downstream Side of Intake.

Taken December 4th, 1922.

Note: This picture gives the detail of the downstream side of the three sluiceways and ice booms between the submerged tubes.



intake as a submerged orifice, a mean value of 1.04 for the coefficient "C" is found. Applying this for the full size intake with the quantity, "q", equal to 18,200 cubic feet per second, the loss is found to be 1.32 feet, and for a discharge of 15,000 cubic feet per second it is 0.92 feet.

The loss in the completed Intake as adopted may be computed by considering it to be made up of a loss at the entrance estimated at 0.10 $\frac{v^2}{2g}$ if the tube is rounded, a loss in friction in the tube and in the diffuser, and a loss in divergence in the diffuser. Combining these losses, the total loss for the Intake is found to amount to 0.60 feet for 18,200 cubic feet per second total discharge, and 0.42 feet when the discharge is 15,000 cubic feet per second.

The loss in the Intake need be considered only for those periods when the tubes are in use. At such times the Chippawa level would be lowered about 0.5 feet.

The Welland River Section and the Earth Section of the Canal.

In computing the flow capacity and the other hydraulic characteristics of the channel through earth, namely, in the Welland River and in the upper part of the Canal, it has been assumed that the bottom width is 150 feet with side slopes of two horizontally to one vertically. It should be remembered that the earth section is not the governing factor in the design, since almost any desired hydraulic characteristics may be obtained within the limits of economy by continued dredging and earth excavation. Such betterments may be conducted without stopping the plant. The hydraulic formula used in the calculations is the well-known Kutter formula, $v = C \sqrt{rs}$ where "v" is velocity

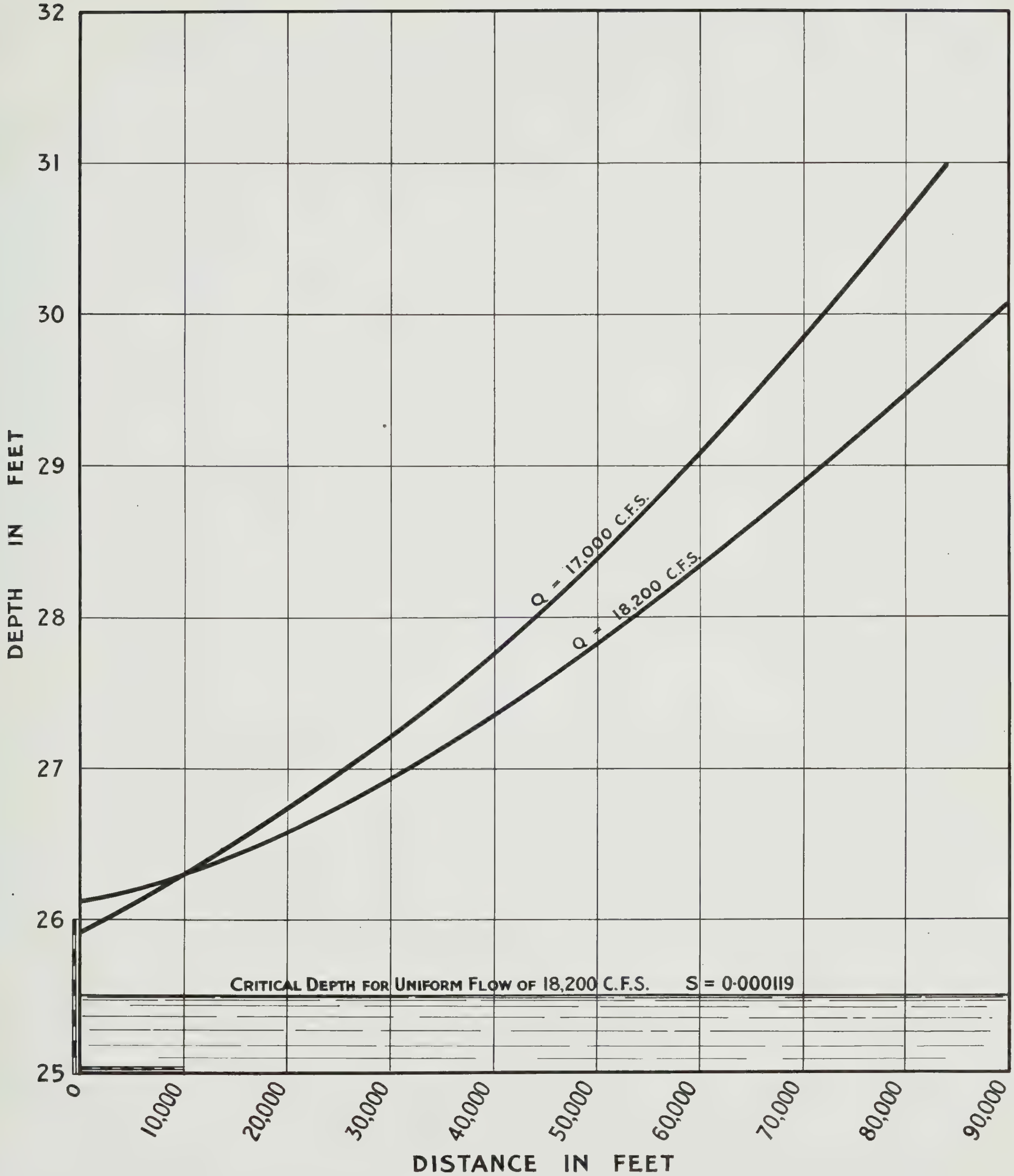
in feet per second, "r" the hydraulic radius in feet and "s" is the slope. The value of the co-efficient "C" is expressed in terms of "r", "s" and the degree of roughness of the surface over which the water is flowing, and may be expressed as follows:

$$C = \frac{\frac{1.811}{n} + 41.65 + \frac{0.00281}{s}}{1 + \frac{n}{\sqrt{r}}\left(41.65 + \frac{0.00281}{s}\right)}$$

in which "n" is an abstract number whose value depends only upon the roughness of the surface. For concrete lined canals "n" is taken as between 0.011 and 0.014, for clean canals in firm gravel it is 0.020 and for canals and rivers in bad order and having a rough bottom with stones and weeds it may be as much as 0.035 or 0.040.

For the earth section of the Canal "n" is taken as 0.035.

The maximum discharge for which the computations were made is 18,200 cubic feet per second. Similar calculations for a discharge of 17,000 cubic feet per second have also been made. Since neither of these amounts is that for which the surface of the water is parallel to the bed of the channel, the Chezy equation of flow does not apply, and it was therefore necessary to solve the surface curve step by step in accordance with the general equation of flow indicated on the diagram on page D-29. This diagram also shows the profile of the surface in the Welland River and earth section of the Canal for the two assumed discharge rates. The results of the computations are shown on the diagram which indicates the surface slope when the flow is 17,000 cubic feet per second, and, when it is 18,200 cubic feet per second, with the slope of the bottom constant at 0.000119. From an examination of this diagram it will



GENERAL EQUATION OF FLOW FROM WHICH THE ABOVE CURVES WERE PLOTTED:

$$l = \frac{(d_2 - d_1) - \frac{q^2}{2g}\left(\frac{1}{a_1^2} - \frac{1}{a_2^2}\right)}{i - \frac{q^2}{C^2 a^2 r}}$$

WHERE
l = LENGTH OF CHANNEL
d = DEPTH AT ENDS OF LENGTH "l"
q = DISCHARGE
a = AREAS OF CROSS-SECTION AT ENDS OF LENGTH "l"
r = HYDRAULIC RADII FOR END SECTIONS

HYDRO-ELECTRIC INQUIRY COMMISSION
W. D. GREGORY, CHAIRMAN
QUEENSTON-CHIPPAWA POWER DEVELOPMENT

PROFILE OF WATER SURFACE IN WELLAND RIVER

Toronto, July 30th, 1923. Made by S.R.W., Checked by W.D.A.

WALTER J. FRANCIS & COMPANY
CONSULTING ENGINEERS

be seen that for a depth of, say, 27 feet at a given point, the depth is 27.9 feet at another point 20,000 feet farther downstream from the given point for a flow of 18,200 cubic feet per second. The critical depth for a uniform flow of 18,200 cubic feet per second is 25.5 feet, being the depth for which the surface is parallel to the bed of the stream. At any depth greater than 25.5 feet the surface is flatter, and at any less depth it is steeper for a discharge of 18,200 cubic feet per second. It is therefore evident from the diagram that the depth of the stream in the Welland River and in the earth section of the Canal increases from the Intake towards the rock section, or, in other words, the surface is on a flatter slope than the bed of the Canal.

Transition from Earth Section to Rock Section.

The next change in the canal section at which hydraulic losses must be considered occurs between Station 60 and Station 65, canal chainage, where the section changes from earth to rock. In calculating the losses the transition is assumed to be a regular change from the earth cross-section to that of the rock section. It is considered to be paved or otherwise protected, and for the purposes of computing friction loss the co-efficient for the Kutter formula is taken as 0.035, the same as for the regular earth section. Stage relations for Elevation 65 as compared with various assumed stages at Station 60 have been determined by the accepted hydraulic formulae, but as they are relatively small they are not included as a separate diagram herein.

be seen that for a depth of, say, 27 feet at a given point, the depth is 27.9 feet at another point 22,000 feet farther downstream from the given point for a flow of 15,000 cubic feet per second. The critical depth for a uniform flow of 15,000 cubic feet per second is 23.8 feet, being the depth for which the surface is parallel to the bed of the stream. At any depth greater than 23.8 feet the surface is flatter, and at any less depth it is steeper for a discharge of 15,000 cubic feet per second. It is therefore evident from the diagram that the depth of the stream in the [illegible] River and in the earth section of the Canal increases from the [illegible] towards the rock section, or, in other words, the surface is on a flatter slope than the bed of the Canal.

COPY

Transition from Earth Section to Rock Section.

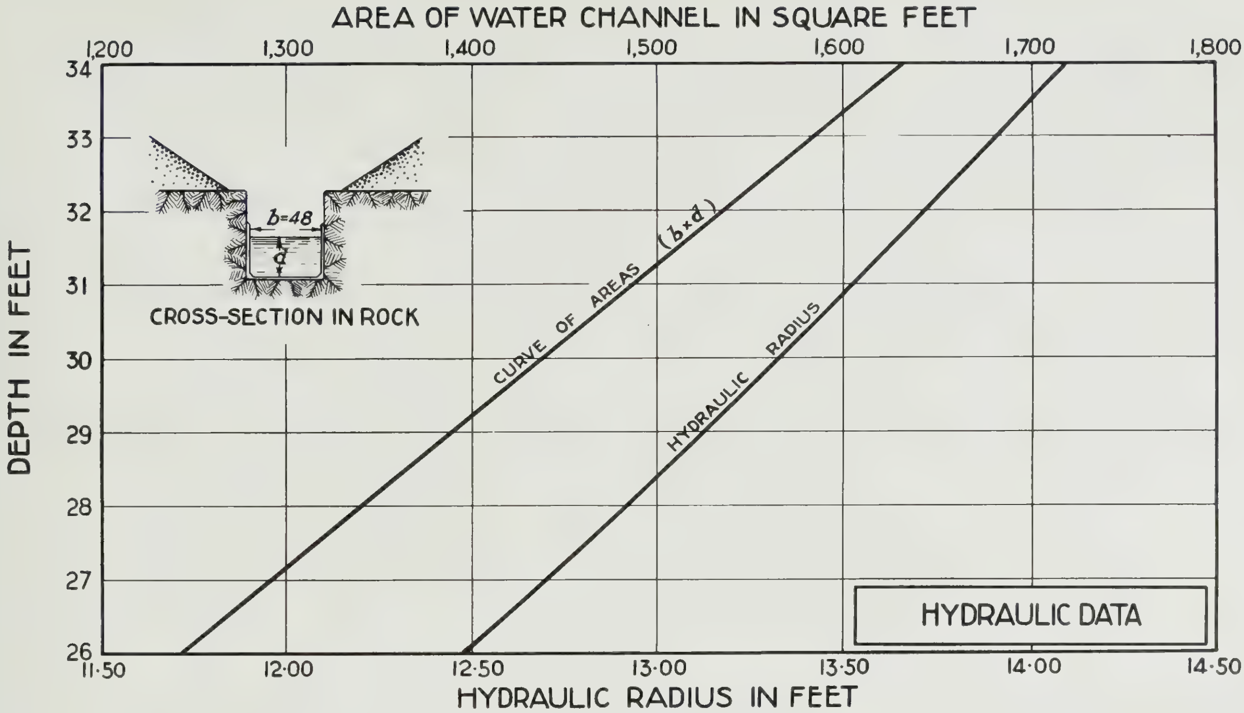
The next change in the canal section at which hydraulic losses must be considered occurs between Station [illegible] and Station [illegible], where the section changes from earth to rock. In calculating the losses the transition is assumed to be a regular change from the earth cross-section to that of the rock section. It is considered to be paved or otherwise protected, and for the purposes of computing friction loss the co-efficient for the Kutter formula is taken as 0.025, the same as for the regular earth section. Water elevations at Station [illegible] as compared with various assumed stages at Station [illegible] have been determined by the accepted hydraulic formulas, but as they are relatively small they are not included as a separate diagram herein.

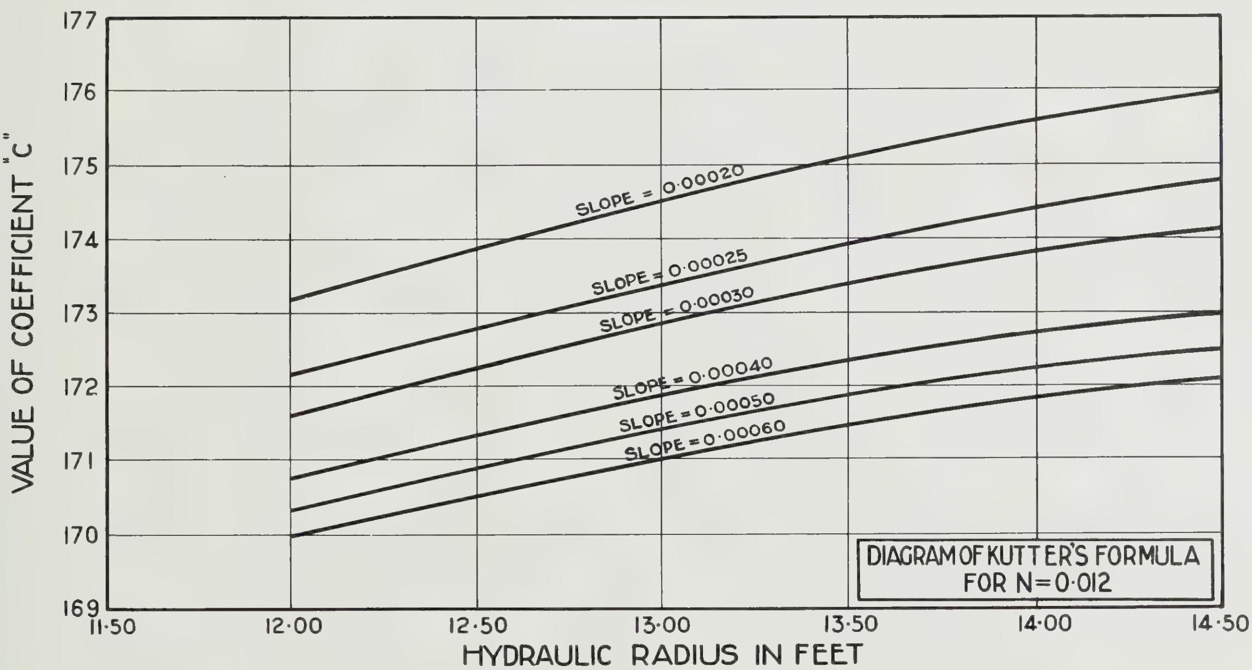
The Rock Section.

The rock section of the Canal is the governing factor in the power available at the Queenston-Chippawa Power Development because betterments are not economically feasible, and a close study of its hydraulic characteristics has been made. The computation of the surface curves for a flow of 17,000 cubic feet per second and of 18,200 cubic feet per second have been made in a manner similar to that described for the earth section of the Canal and for the Welland River. The hydraulic elements for the rock section are shown on the diagram included as page D-32, and the results of the computation for the surface curves for the two assumed rates of flow are shown on the diagram included as page D-33. The depth of stream is plotted vertically, and the distance along the channel horizontally, the same as for the earth section, and it will be noted that the characteristics of the curves are quite different from those in the earth section, the depth decreasing with the distance. The increasing steepness of the slope of the surface curves as the distance increases accounts for the magnified effect of the change of water elevation at Chippawa on the water elevations at the Screen House. An examination of the surface curves for the flows indicated shows the relation of the depth of the stream to the surface between any two chosen points. For example, with a flow of 18,200 cubic feet per second where the depth of the stream is 34 feet, at a point 20,000 feet farther down the Canal the depth is seen to be 31.5 feet.

Whirlpool Trapezoidal and Transition Sections.

From the rectangular rock section at Station 329+50 a transition section





HYDRO-ELECTRIC INQUIRY COMMISSION

W. D. GREGORY, CHAIRMAN

QUEENSTON-CHIPPAWA POWER DEVELOPMENT

HYDRAULIC ELEMENTS FOR ROCK SECTION OF CANAL

Toronto, July 30th, 1923. Made by G.H.D. Checked by W.D.K.

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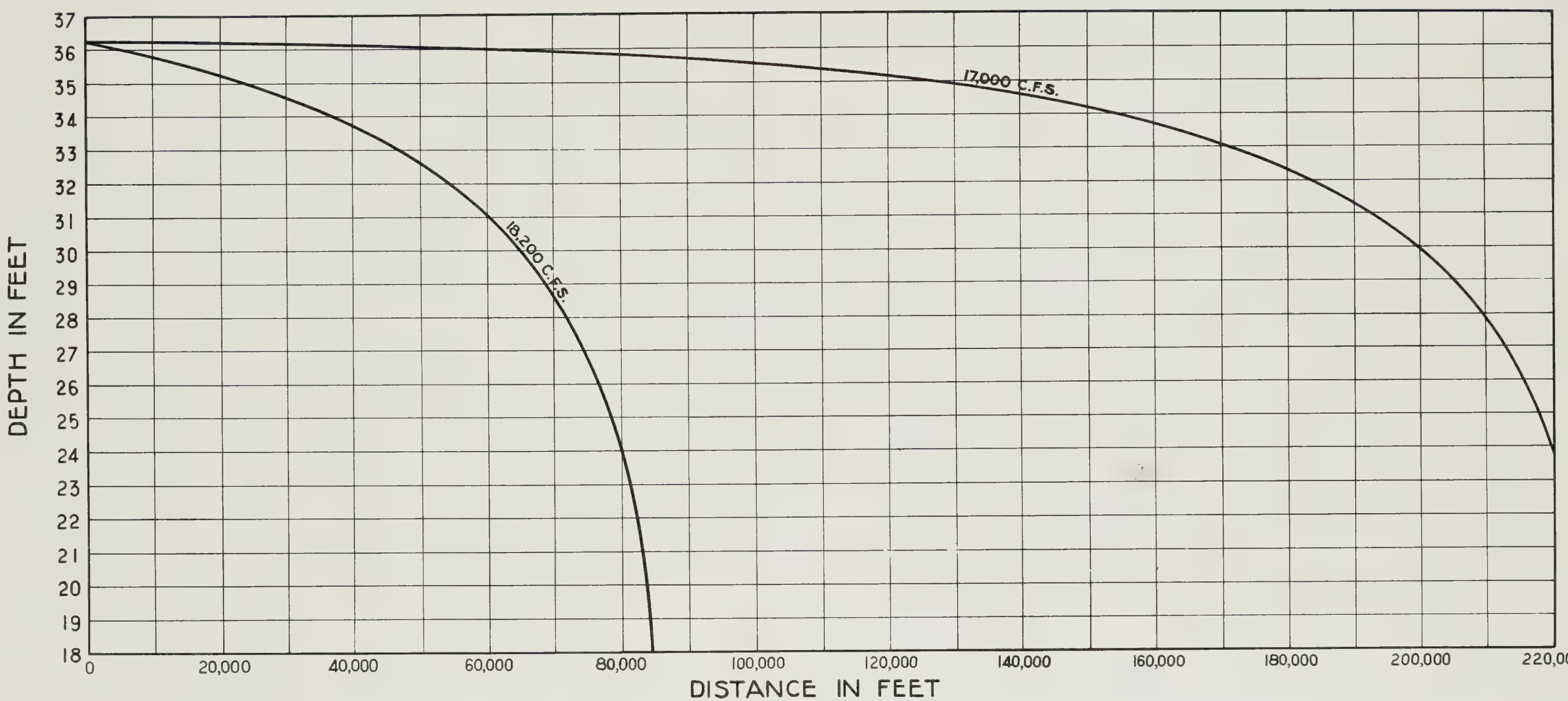
(D-33)



HYDRO-ELECTRIC INQUIRY COMMISSION

W. D. GREGORY, CHAIRMAN

QUEENSTON-CHIPPAWA POWER DEVELOPMENT

WATER SURFACE PROFILES IN ROCK SECTION

Toronto, July 30th, 1923. Made by [illegible], Checked by [illegible]

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extends to Station 322+50, in which the form of the water channel changes from the rectangular cut in the rock to the trapezoidal section of the Whirlpool Section. At Station 348+50 a similar transition section leads from the trapezoidal form back to the rectangular form at Station 354+00.

The losses through these three sections have been computed in detail using the same basis for the transition sections as already described for that between Station 60 and Station 65. The relation of the stages at Station 332+50 and at Station 348+50 have also been computed and plotted in the diagrams, but for the sake of brevity these details are not included herein as separate diagrams.

Forebay.

When the water of the Canal reaches the entrance to the Forebay at Station 452+87 the velocity is high and there is considerable kinetic energy stored in it. The duty of the Forebay is two-fold. One duty is to discharge the water from the narrow rock section of the Canal across the five hundred-foot width of the penstock entrances. This duty by itself would not require any very special form of forebay, but the other duty required of it necessitates a special forebay design. This second duty is that of redeeming from the moving water as much as possible of its kinetic energy or velocity head and changing it back into potential energy or static head. The method of accomplishing this is to lower the velocity of the water gradually by a corresponding widening of the channel. This has been done at the Forebay by constructing at its entrance a triangular or wedge shaped mass of concrete known as a diffuser.

At the Forebay, therefore, there is a loss of head due to the friction loss and to the divergence of the stream lines as the section of the channel is enlarged, and there is also a change or a regaining of the head from the form of velocity head to the static head due to the decrease in the velocity of the water. Considering these changes, the relation between the stage of the water at the throat, Station 452+87, and at the Screen House has been calculated from the usual hydraulic formulae, taking into account the fact that after the water has passed the diffuser there is still a certain amount of velocity head in it and assuming that one-half of this residue is saved in the conversion to static head.

In order to determine the loss due to the divergence, experimental data on divergence tubes had to be used as there are no such data available for divergence loss in open channels. The experiments of Dr. A. H. Gibson were used in making the calculations, but a factor of 25 per cent. was used, to be conservative, instead of 18 per cent. deduced from the experiments of Dr. Gibson.

Canal tests made during the past few months have demonstrated the fact that the calculated results have been exceeded, and that about 80 per cent. of the velocity head has been regained in the Forebay.

<u>Miscellaneous Losses.</u>

The losses due to contraction of the stream at bridge piers, and the loss due to curvature at the bends of the Canal have been omitted in the above mentioned computations because they are quite small in amount and because there are no reliable experimental constants for calculating them.

Hydraulic Studies of the Canal.

Relation of the Water Stages at Chippawa and at the Screen House.

By using the complete series of calculations and diagrams referred to herein showing the various losses occurring in the water channel from Chippawa to the Screen House, it is possible to determine the elevation of the water surface at the Screen House corresponding to any given elevation at Chippawa when the discharge is 17,000 cubic feet per second, and also 18,200 cubic feet per second.

For a discharge of 18,200 cubic feet per second with the stage at Chippawa taken as Elevation 561.00 just inside the Intake, the depth is 28.50 feet. On the diagram showing the surface curve in the earth section, if a distance representing 27,000 feet be measured off, Station 60+00 on the Canal, the end of the earth section, is reached, and the depth is found to be 30.10 feet at that point. Similarly, using the calculations for the stage at Station 65, the elevation is found to be 557.37, corresponding to 559.39 for the stage at Station 60+00, and the depth at Station 65+00 is 35.04 feet. To find the stage at Station 329+50 the diagram for the surface curve in the rock section is used, and from the point on the curve where the depth is 35.04 feet a horizontal distance of 26,400 feet is measured, where a point is found the depth at which is seen to be 32.65 feet, making the stage at that point Elevation 549.39. Using the other diagrams and calculations in a similar manner, the stage may be carried on down the Canal to the Screen House where the stage is about Elevation 546.90.

Using the above method, a diagram has been plotted showing the relation

of the water stages at Chippawa and at the Screen House for a flow of 17,000 cubic feet per second, and of 18,200 cubic feet per second. This diagram is included as page D-38 and from it the relation of the stage at Chippawa with that at the Screen House may be found. For example, when the stage at Chippawa is at Elevation 560.25, the stage at the Screen House is at Elevation 543.00 for a flow of 18,200 cubic feet per second, while for a flow of 17,000 cubic feet per second the stage at the Screen House would be about Elevation 548.00. Similarly, if the elevation of the water at Chippawa is at any other figure between Elevation 559.50 and Elevation 562.00, the elevation of the water at the Screen House can be found by measuring along the horizontal line to the intersection with the curves. From the curves on page D-38 it is evident that when the level at Chippawa is at Elevation 560.00 the variation in the stage at the Screen House is about six times as much as it is at Chippawa. The diagram included as page D-39 has been prepared, showing the profile of the water channel from the Niagara River to the Screen House. On this profile are plotted the profiles of the water surface which would result from the stage of the water surface at Chippawa being respectively at Elevations 560.00, 561.00 and 562.00 on the Hydro-Electric Power Commission datum, and the discharge being 18,200 cubic feet per second. The three lines illustrate the increased slope in the surface through the rock channel as the depth decreases.

Flow Tests in the Canal.

Six separate flow tests in the Canal made during July, 1923, using standard automatic recording float gauges with the flow ranging between 6,000 and 7,000

of the water stage at Culpepper and at the Screen House for a flow of 17,000 cubic feet per second, and of 14,[illegible] cubic feet per second. This diagram is included on page D-[illegible] and from it the relation of the stage at Culpepper with that at the Screen House may be found. For example, when the stage at Culpepper is at Elevation [illegible], the stage at the Screen House is at Elevation [illegible] for a flow of 14,[illegible] cubic feet per second, while for a flow of 17,[illegible] cubic feet per second the stage at the Screen House would be about Elevation [illegible]. Similarly, if the elevation of the water at Culpepper is at any other figure between Elevation [illegible] and Elevation [illegible], the elevation of the water at the Screen House can be found by measuring along the horizontal line to the intersection with the curve. From the curves on page D-28 it is evident that when the level at Culpepper is at Elevation [illegible] the variation in the stage at the Screen House is about [illegible] times as much as it is at Culpepper. The diagram included on page D-[illegible] has been prepared, showing the profile of the water channel from the [illegible] to the Screen House. On this profile are plotted the profiles of the water surface which would result from the stage of the water surface at Culpepper being respectively at Elevations [illegible], [illegible] and [illegible] on the [illegible] gauge, and the discharge being 18,200 cubic feet per second. The curves [illegible] illustrate the increased slope in the surface through the rock channel at the [illegible].

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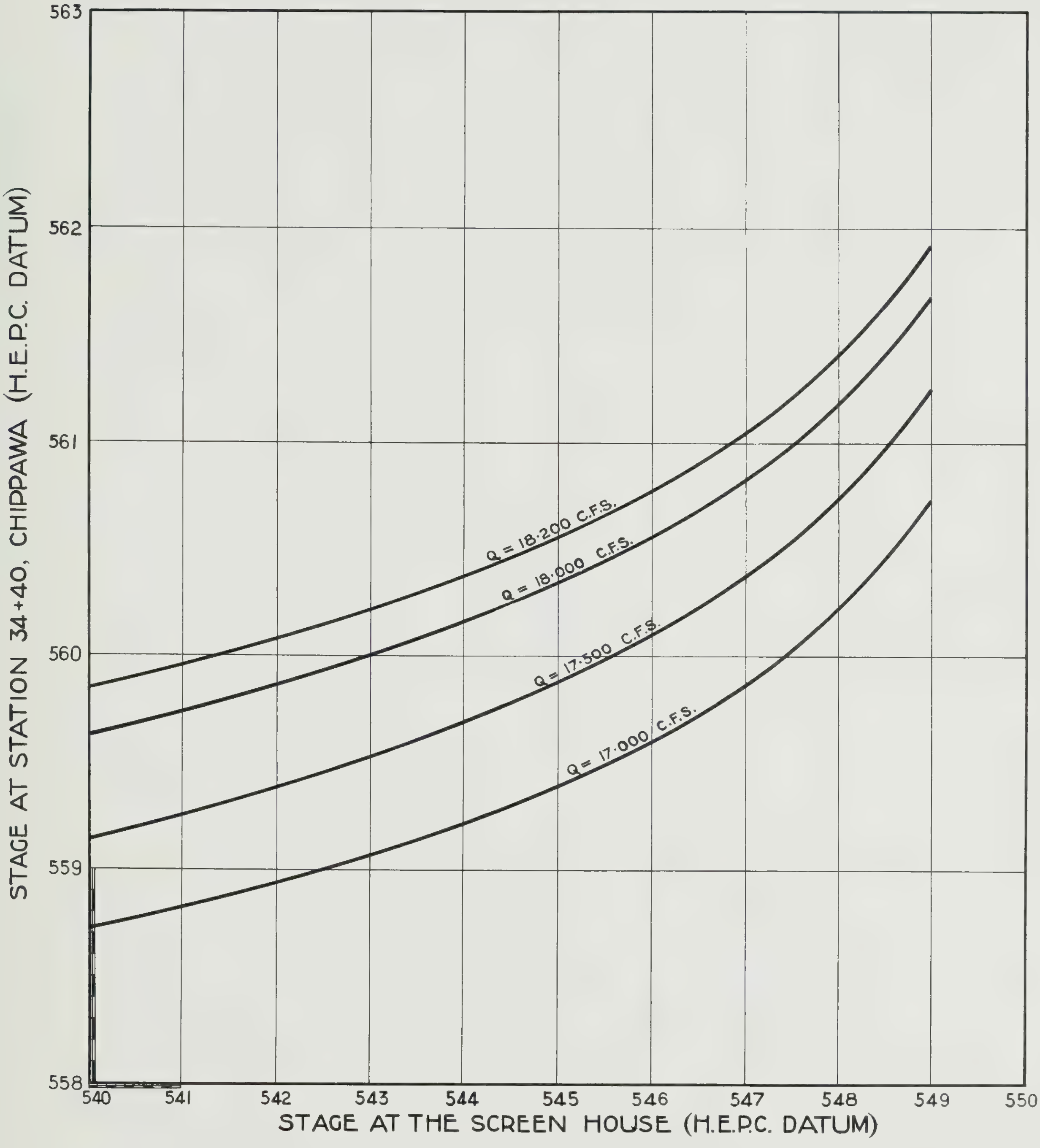
Flow Tests in the Canal.

Six separate flow tests in the canal were made during July, 1925, using standard [illegible] recording float gauges with the flow ranging between [illegible] and [illegible]



HYDRO-ELECTRIC INQUIRY COMMISSION

W. D. GREGORY, CHAIRMAN

QUEENSTON-CHIPPAWA POWER DEVELOPMENT

RELATION OF WATER STAGES AT CHIPPAWA AND SCREEN HOUSE

Toronto, July 30th, 1923. Made by C.H.S., Checked by

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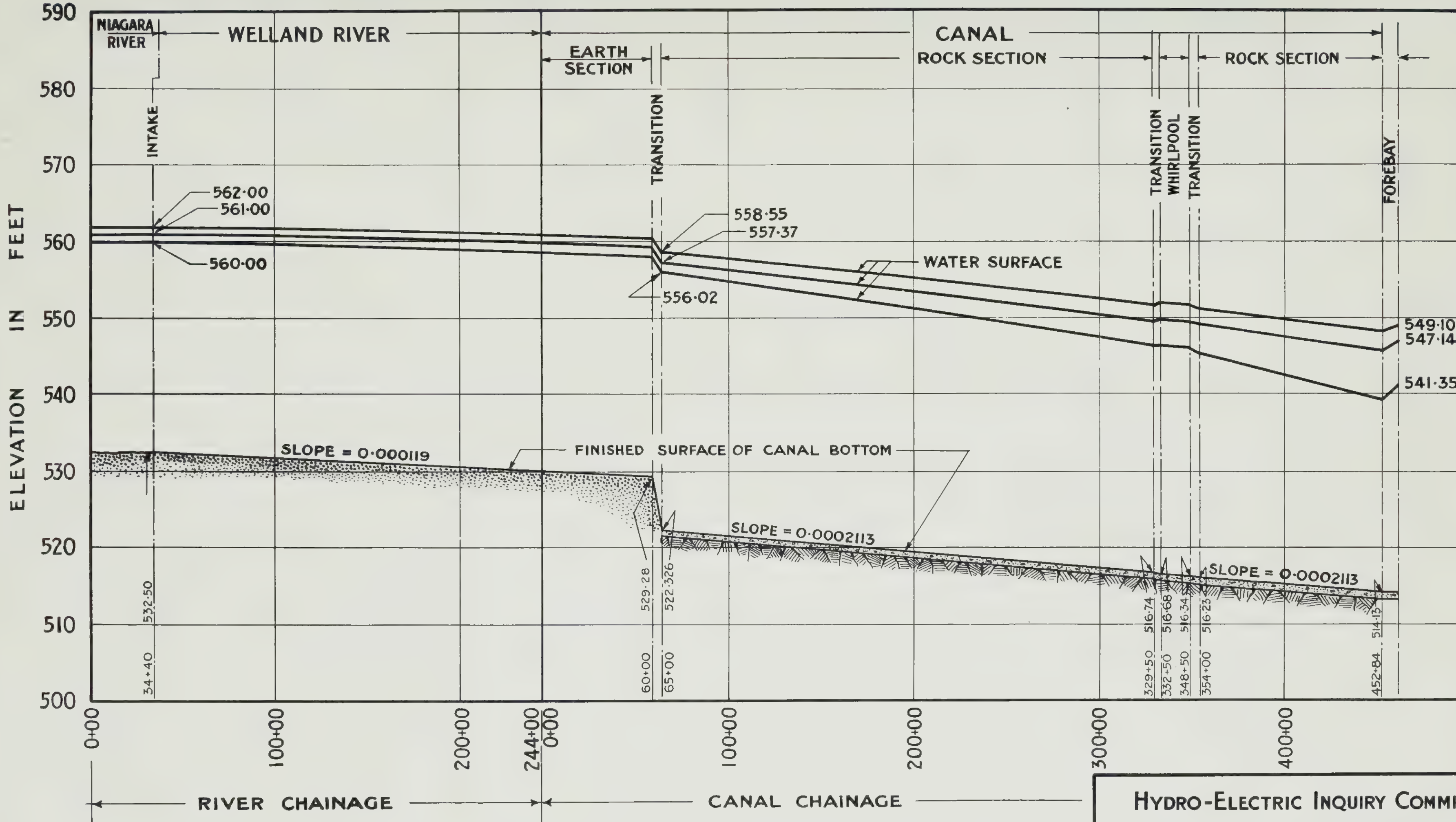

Hydro-Electric Inquiry Commission

W. D. Gregory, Chairman

Queenston-Chippawa Power Development

PROFILE OF WATER CHANNEL, NIAGARA RIVER TO SCREEN HOUSE

Toronto, July 30th, 1923. Made by S.R.W., Checked by W.J.F.

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Consulting Engineers

cubic feet per second, gave results which when applied for a flow of 18,000 cubic feet per second were practically an absolute check on the above calculations. Considering that it is known that the concrete floor of the Canal is littered with rock spoil and silt collection as a result of construction operations subsequent to the filling of the Canal with water in December, 1921, and that the assumed condition of a clean concrete floor has not yet been attained, it would appear that the estimated flow capacity of the Canal will be exceeded under normal completed working conditions. The figures of the derived results of the tests are given in the table on page D-41.

Surge Action in the Canal. COPY

A study has been made to determine the effect of surge on the water surface in the Canal.

The general theory of the surge produced by the sudden closing down of the plant may be described as follows: The plant is assumed to be drawing 18,200 cubic feet of water per second, and to be in normal operation. The load is suddenly thrown off, and the gates of the turbines are closed. The phenomena to be expected are as follows,- The down-coming water in the Forebay would first tend to rise up against the upstream wall of the Screen House substructure, forming a "seiche", or fresh water tide, due to the conversion of velocity head into static head. The down-coming water in the Canal would tend to accumulate against the former seiche and a wave would result which would travel up the Canal. If the relative velocity of the wave and of the current of the water in the Canal is greater than $\sqrt{gh}$, where "h" is the depth of the Canal,

COPY

Discharge during tests, cubic feet per second ....

[illegible] from Station [illegible] to station [illegible], feet ....

Depth at Station [illegible], feet ....

Depth at Station 432+02, feet ....

Hydraulics: [illegible] head, in feet, in rock section, Station [illegible] to Station 432+00 for the same depths as above and for a discharge of 10,000 cubic feet per second ....

Increase in velocity head, in feet, from Station [illegible] to station [illegible]

Total fall of water surface in rock section, in feet ....

Assumed elevation at beginning of rock section, station [illegible] ....

Calculated elevation of water surface at station [illegible] ....

Rise of water surface in [illegible], in feet, due to recovery of velocity head ....

Resultant elevation of water surface at [illegible] ....

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QUEENSTON-CHIPPAWA

Table of Water Elevations

Derived from the Canal

Data

Discharge during tests, cubic feet per second ....

Friction head from Station 213+34 to Station 452+00, feet ....

Depth at Station 213+34, feet...

COPY

Depth at Station 452+00, feet ....

Equivalent friction head, in feet, in rock section, Station 65+00 to Station 452+00 for the same depths as above and for a discharge of 18,000 cubic feet per second ....

Increase in velocity head, in feet, from Station 65+00 to Station 452+00 ..

Total fall of water surface in rock section, in feet ....

Assumed Elevation at beginning of rock section, Station 65+00 ....

Calculated Elevation of water surface at Station 452+00 ....

Rise of water surface in Forebay, in feet, due to recovery of velocity head

Resultant elevation of water surface at Screen House ....

wer DEVELOPMENT

the Screen House

sts of July, 1923

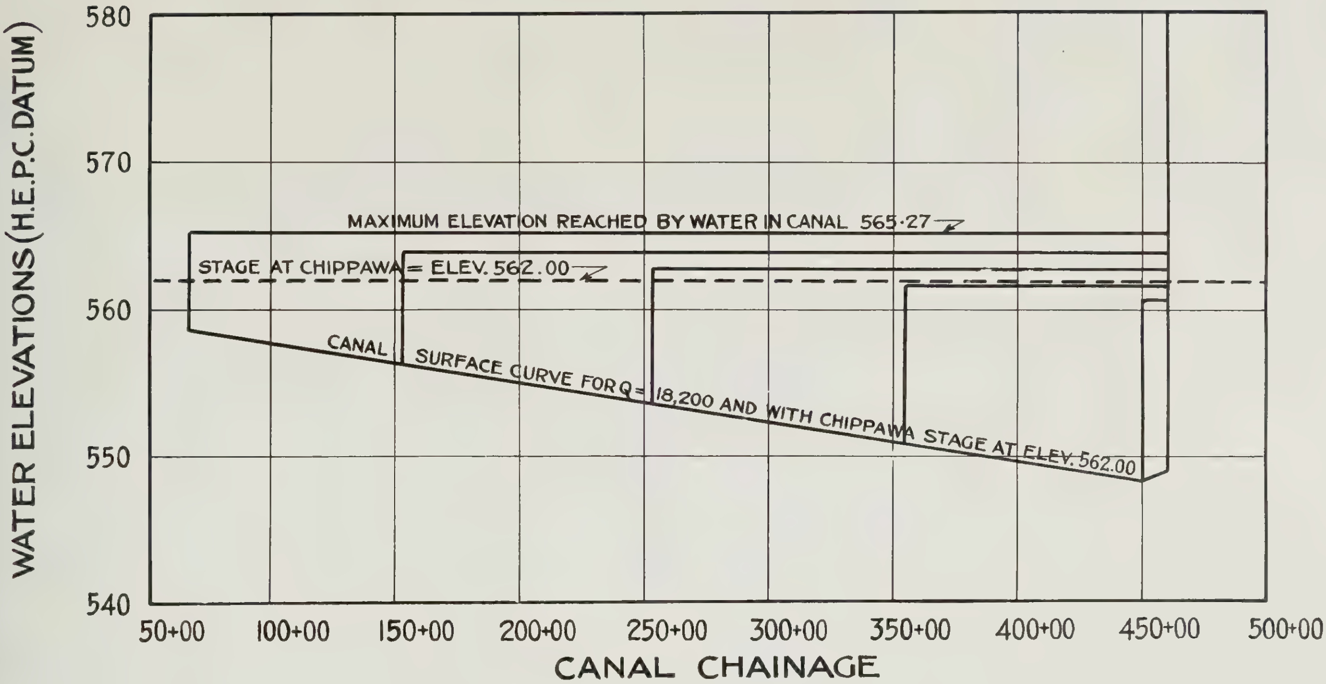
est No. 1 July 6th 00 to 3.00 p.m.	Test No. 2 July 6th 3.00 to 4.00 p.m.	Test No. 3 July 23rd 2.40 to 3.40 p.m.	Test No. 4 July 24th 2.00 to 3.00 p.m.	Test No. 5 July 24th 3.00 to 3.50 p.m.	Test No. 6 July 28th 10.00 to 10.50 a.m.
6,228	6,223	6,106	6,820	6,818	5,943
1.11	1.12	1,013	1.357	1.357	1.018
32.21	32.18	32.84	31.98	32.11	32.52
36.19	36.15	36.92	35.73	35.86	36.55
15.04	15.18	14.28	15.36	15.36	15.18
1.82	1.87	1.55	1.94	1.94	1.87
16.86	17.05	15.83	17.30	17.30	17.05
556.23	556.23	556.23	556.23	556.23	556.23
539.37	539.18	540.40	538.93	538.93	539.18
2.78	2.82	2.56	2.87	2.87	2.82
542.15	542.00	542.96	541.80	541.80	542.00

the wave will be of the standing type, but will in reality have its front moving up the Canal. The down-coming water in the Canal will continue to accumulate against the wave and tend to lift its level. Considering these phenomena, it is seen that two conditions arise which must be in harmony, namely, that the velocity of the wave crest in an upstream direction must be sufficient for the down-coming water to fill up the wedge of the channel beneath the crest of the wave and the former surface of the stream, and that the velocity of this wave must agree with that found for a standing wave of such a height that the relative velocity of the stream and the wave is made up of the velocity of the wave itself added to the velocity of the water in the stream.

We have plotted a diagram showing the surge action in the Canal for the assumed conditions. The diagram is included as page D-43.

On the diagram will be seen a sketch illustrating the theory from which the equations for the surge resulting from a complete shut down have been derived.

In the illustrative small diagram are given the factors in algebraic form which enter into the computations. Assuming that in the time "t" from the instant of shut down the wave has reached a position "Z", and that it is "X" in height, with the rate of discharge in the Canal "q", then in the time "t" the total discharge is "q" x "t". This must equal the mass of the water shown above the line representing the slope under normal flow conditions, for which the volume is as follows, "b" being the width of the Canal in feet, and "A" being the area of the Forebay, namely 413,000 square feet, "t" being in seconds,



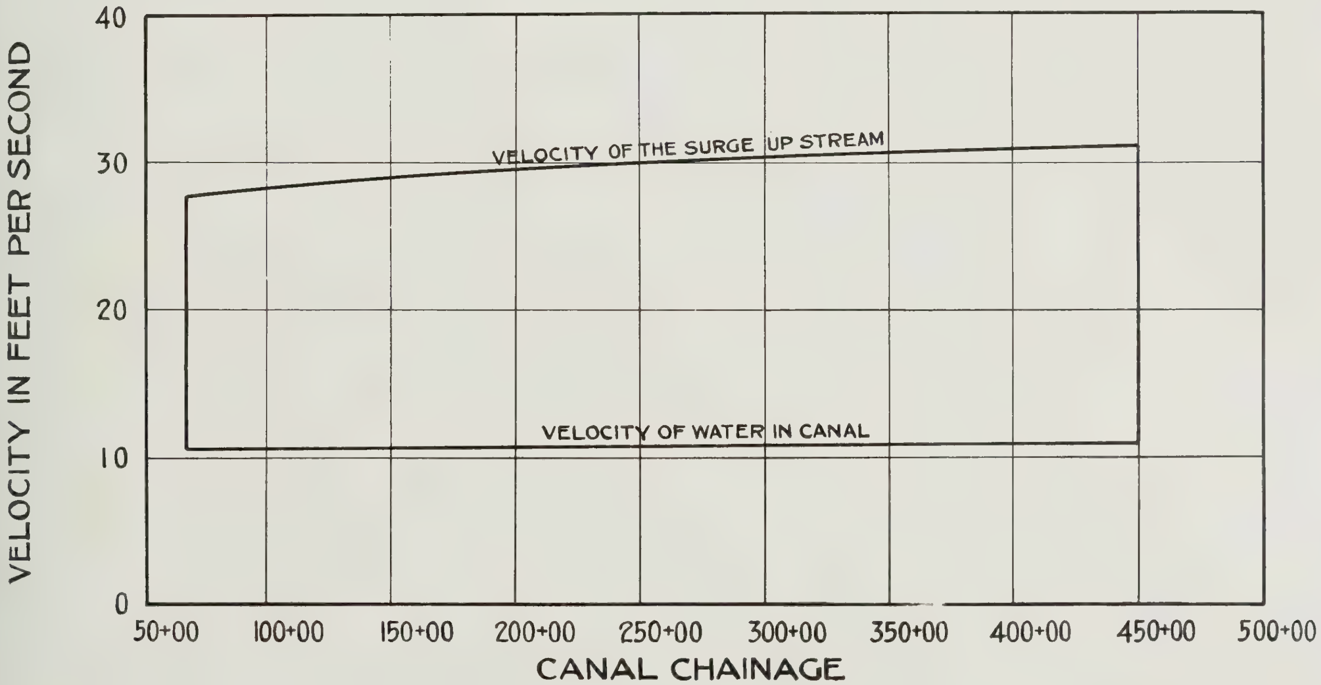
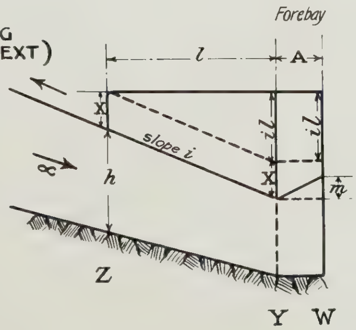


DIAGRAM ILLUSTRATING SURGE ACTION (SEE TEXT)



HYDRO-ELECTRIC INQUIRY COMMISSION

W. D. GREGORY, CHAIRMAN

QUEENSTON-CHIPPAWA POWER DEVELOPMENT

CANAL SURGE DIAGRAMS

Toronto, July 30th, 1923. Made by G.G.B. Checked by W.O.A.

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"X" being in feet, and "q" being in cubic feet per second:

$$q \times t = (b\ l\ X) + (\tfrac{1}{2}\ i\ b\ l^2) + \left\{A\ (X + i\ l)\right\} - (\tfrac{1}{2}\ Am)$$

In this expression "t", "l" and "X" are variables, so, differentiating by calculus with regard to "t" as an independent variable, we get

$$q \times dt = (b\ l\ X \times dX) + (BX \times dl) + (ibl \times dl) + (A\ X \times dX) + (iA \times dl)$$

Solving for $\frac{dl}{dt}$, since this is the velocity of the wave with regard to the bed of the channel, we find that

$$\frac{dl}{dt} = \frac{q - (bl + A)\frac{dx}{dt}}{bx + ibl + Ai} \qquad \text{(Equation 1.)}$$

If we call "v" the relative velocity of the wave front with regard to the water in the stream, "p" being the stream velocity, we have

$$v = \frac{dl}{dt} + p \qquad \text{(Equation 2.)}$$

The expression for the height "X" of a standing wave in a stream of depth "h" and where the velocity of the wave with regard to the water is "v" is as follows:

$$X = \sqrt{\frac{h^2}{4} + \frac{2\ h\ v^2}{g}} - \frac{3}{2}h \qquad \text{(Equation 3.)}$$

Therefore, by assuming a value of "X", and thus determining from Equations 1 and 2 a value of "v", Equation 3 is used for a test and check to see if the assumed value of "X" is correct. Trial and error methods must be used for the solution of these equations, but, while tedious, they are accurate.

The general formulae underlying the computations have been applied to the assumed case, and the following table has been prepared showing the effect of the surge at various points on the Canal.

Table of Surge Produced

Flow shut off = 18,200 cubic feet per second
Water Stage at Chippawa = Elevation 562.00

Station	452+87	354+00	252+87	152+87	65+00	60+00
Elevation	548.09	551.00	553.63	559.29	558.55	560.57
Depth = h	33.96	34.77	35.28	35.82	36.22	31.28
Water velocity = p	11.18	10.92	10.75	10.56	10.46	-
Slope = i	0	.00029	.000278	.000273	.000271	-
Length = l	0	9,887	20,000	30,000	38,800	-
Surge height = X	12.20	10.50	9.12	7.78	6.72	-
Surge velocity	31.07	30.50	29.50	29.00	27.52	-
Time = t	-	325	335	340	1311	-
Relative velocity, surge to water = v	42.25	41.12	-	39.56	37.96	-
dx / dt	-	.00554	-	.00394	-	-
Check of X by Eq. 3	12.60	10.70	-	7.90	6.50	-
Elevation of surge	560.59	561.50	562.75	564.07	565.27	-

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From the above figures and the diagram on D-43 it is seen that the surge starts at the entrance of the Forebay, where it is 12.20 feet in height, and runs to the beginning of the rock section of the Canal at Station 65+00 where the surge is reduced to 6.72 feet in height. At this point the water surface is at Elevation 565.27 or 3.27 feet above the stage at Chippawa.

As the wave enters the transition section the relative velocity of the wave and the water is reduced below the critical value and the comparatively steep wave form breaks down into a long flat swell or slope when the water which has accumulated in the Canal runs out into the river section. Following this, a wave of depression or rather of subsidence runs back down the Canal towards the Forebay.

The first steep wave takes about 22 minutes to travel the distance of 38,800 feet between the Forebay and the beginning of the rock section. In the

analysis the change in the form of the water channel at the Whirlpool was not considered, as the effect in any case would be slight and the calculations would be needlessly complicated.

Summing up the above discussion, it may be concluded that, under the very severe conditions assumed, the surge effect would likely raise the water surface about three feet above the Chippawa level. As it is unlikely that a complete shut down may ever occur quickly, it may be stated that the effect in practice will be to produce much smaller surges than that described above.

The Determination of the Elevation of the Tailrace Waters.

Oswego Gauge.

The levels of the water surface of Lake Ontario have been observed for many years at certain places and by various authorities. The nearest gauge to the Queenston-Chippawa Power Development is at Oswego, New York, where reliable and continuous records are available for all years since 1860, under the United States Lakes Survey. The readings of this gauge have been used in co-relating and checking the gauges placed by the Hydro-Electric Power Commission at the tailrace of the power house and at Queenston dock.

Smeaton's Cove Gauge.

In January, 1915, the Hydro-Electric Power Commission established a gauge at Smeaton's Cove on the west side of the Niagara River about opposite the

northerly end of the site of the Power House. Readings from this gauge are available from February, 1915. This gauge was moved from time to time between October, 1918, and March, 1922, when it was finally located in the tailrace of the Queenston-Chippawa Power Development between Unit No. 1 and Unit No. 2..

Queenston Gauge.

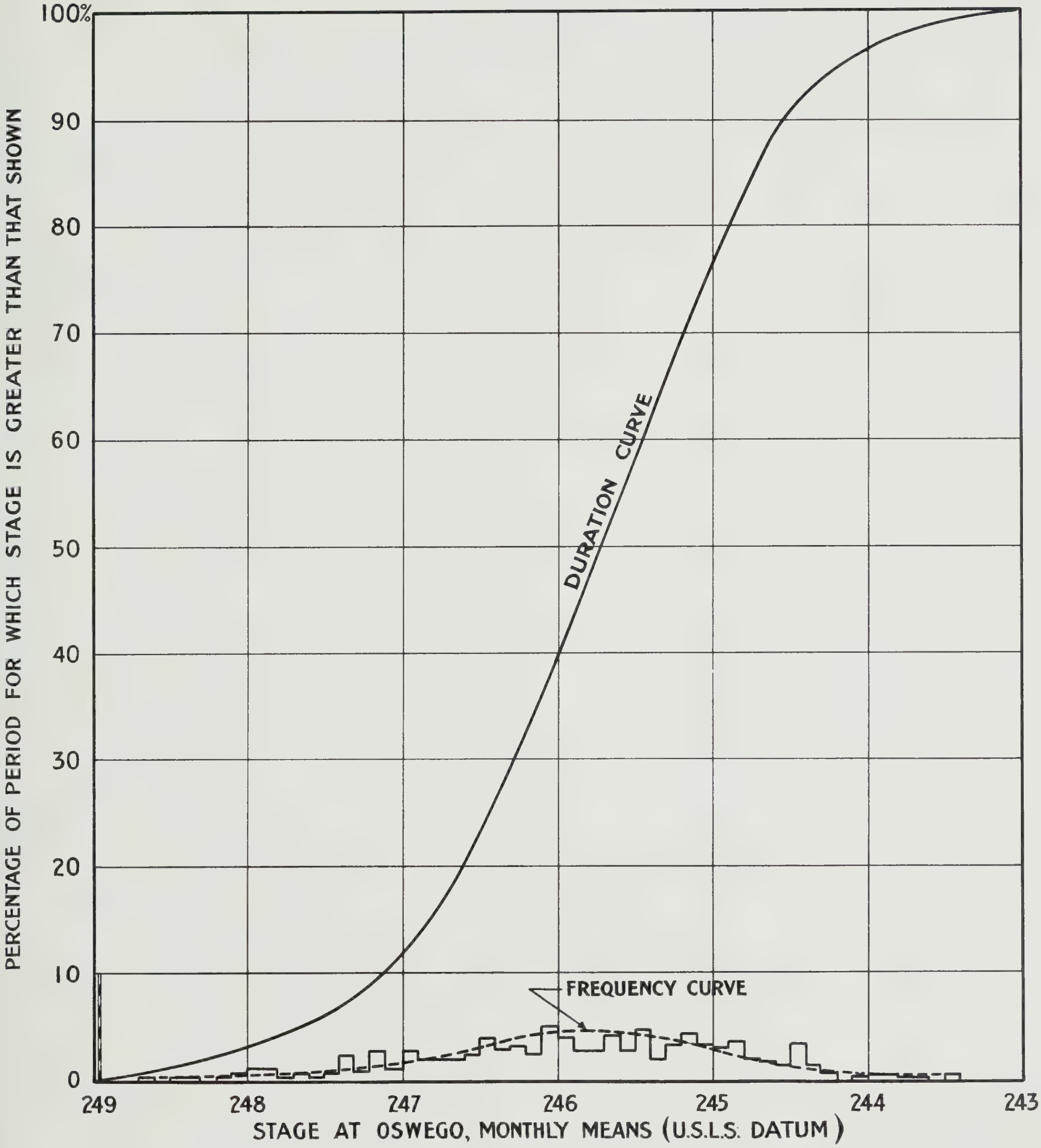
A gauge was located at the Queenston dock, and readings are available from 1917. In July, 1919, the readings are given for a registering gauge at Queenston.

Co-relation of the Tailrace Gauge with Lake Ontario Levels.

In order to determine the relation existing between the stage of the Niagara River at the Power House and also at Queenston and the stage of Lake Ontario, the readings of the Oswego gauge have been taken as the independent variable, and the synchronous monthly mean readings of the various gauges set up in the lower Niagara River have been plotted correspondingly.

The duration and frequency curves for the Oswego gauge are shown on the diagram included as page D-48, embracing the period from 1890 to 1920 inclusive, monthly mean heights being used in all cases. In the gauge relations referred to herein it should be noted that the levels of all gauges in the United States are referred to the United States Lakes Survey Datum, while all the Canadian gauges are referred to the datum of the Hydro-Electric Power Commission of Ontario.

From a study of all the available data a diagram has been plotted showing



HYDRO-ELECTRIC INQUIRY COMMISSION

W. D. GREGORY, CHAIRMAN

QUEENSTON-CHIPPAWA POWER DEVELOPMENT

STUDY OF OSWEGO GAUGE

Toronto, July 30th, 1923. Made by S.R.W., Checked by W.D.A.

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the relation between the water levels of Lake Ontario at Oswego and those at the tailrace of the Queenston-Chippawa power plant. This diagram is included as page D-50, and shows the relation by means of a single line representing a mean tailrace position.

The mean stage at Oswego for the period from 1860 to 1920 inclusive is at Elevation 246.21, U.S.L.S. datum. From the diagram it is seen that the corresponding level at the tailrace is Elevation 247.05, H.E.P.C. datum. The mean value stated by the engineers of the Hydro-Electric Power Commission as being used by them is at Elevation 246.70. From the duration curve it is to be seen that the elevation at Oswego is lower than Elevation 246.21 for 70 per cent. of the period from 1899 until 1921.

If a point be taken such that for 90 per cent. of the time the level would not exceed it, the elevation of the Oswego gauge would be 247.10, giving an elevation at the tailrace of about 248.20.

The Power Output Available at the Queenston-Chippawa Power Development.

General.

The power available at a hydro-electric power plant is a function of the head, the flow and the efficiency. It is expressed by the equation:

$$\text{Horse-power output} = \frac{62.5 \times Q \times H}{550} \times \text{efficiency},$$

in which "Q" is the total rate of flow in cubic feet per second and "H" is the

the relation between the water levels of Lake Ontario at Oswego and those at the tailrace of the [illegible] power plant. This diagram is included as page D-[illegible], and shows the relation by means of a single line representing a mean tailrace position.

The mean stage at Oswego for the period from 1860 to 1920 inclusive is at Elevation 246.11, U.S.L.S. Datum. From the diagram it is seen that the corresponding level at the tailrace is elevation [illegible], U.S.L.S. Datum. The mean value stated by the engineers of the Hydro-Electric Power Commission as being used by them is at Elevation [illegible].70. From the duration curve it is to be seen that the elevation at Oswego is lower than Elevation [illegible] for 50 per cent. of the period from [illegible].

If a point be taken such that for 90 per cent. of the time the level would not exceed it, the elevation of the Oswego gauge would be 247.10, giving an elevation at the tailrace of about 248.20.

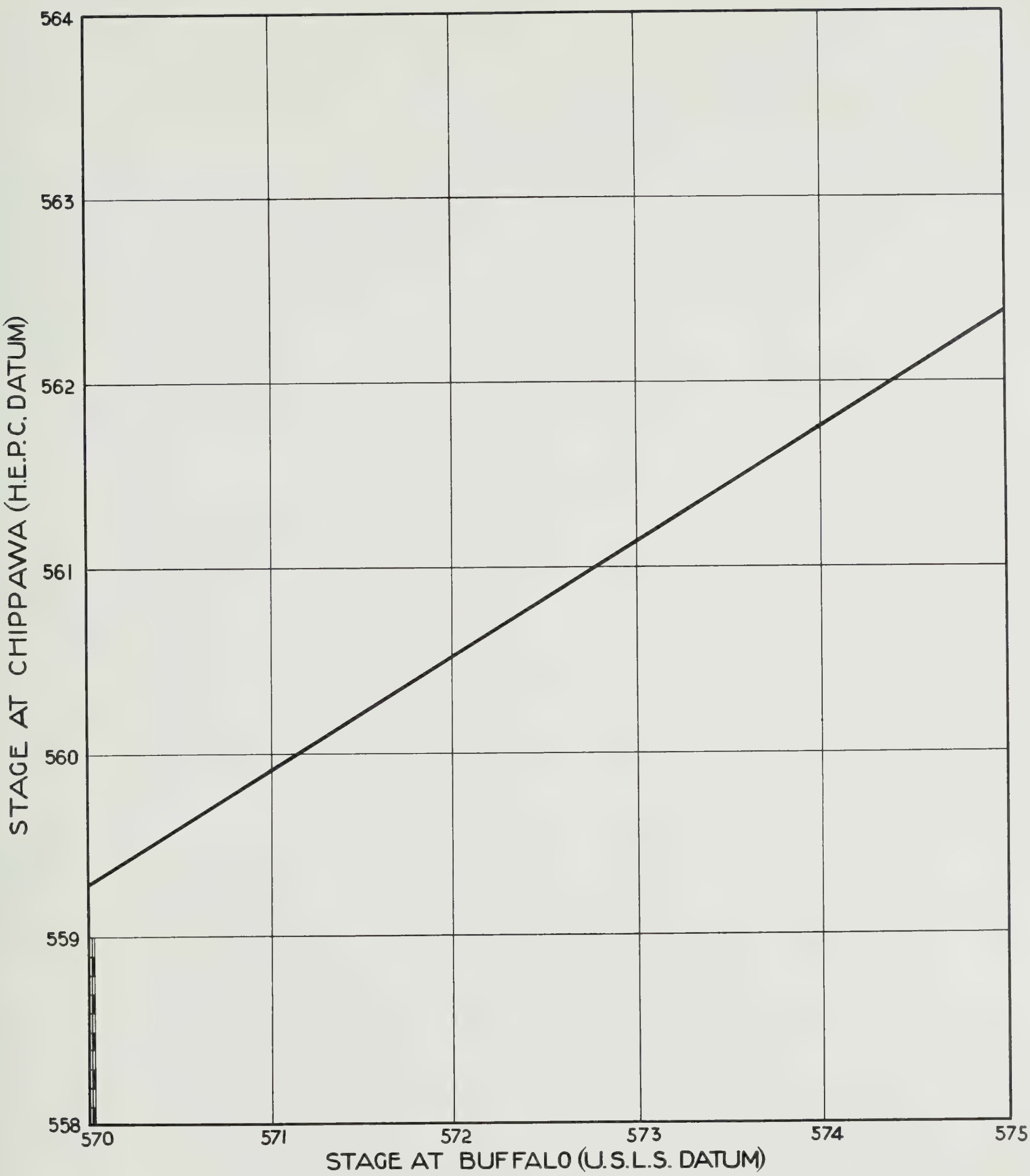
The Power Output Available at the [illegible] Power Development.

General.

The power available at a hydro-electric power plant is a function of the head, the flow and the efficiency. It is expressed by the equation:

$$\text{Horse-power output} = \frac{62.5 \times Q \times H}{550} \times \text{efficiency},$$

in which "Q" is the total rate of flow in cubic feet per second and "H" is the



HYDRO-ELECTRIC INQUIRY COMMISSION

W. D. GREGORY, CHAIRMAN

QUEENSTON-CHIPPAWA POWER DEVELOPMENT

RELATION OF WATER STAGES AT BUFFALO AND CHIPPAWA

Toronto, July 30th, 1923. Made by W.H.D. Checked by W.D.G.

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effective head in feet.

In the Queenston-Chippawa plant tests have demonstrated that over a wide range of output the combined or overall efficiency of both the hydraulic and electric elements of the generating plant is over 90 per cent. so that in this plant the electrical horse-power output at the 12,000-volt bus bars is equal to $0.1022 \times Q \times H$, in which "Q" is the total flow in cubic feet per second and "H" is the difference in level between the surface of the water in the forebay at the Screen House and the mean level in the tailrace. The tests are referred to later in detail.

In order to determine the electrical horse-power output of the Queenston-Chippawa plant for a particular flow, the elevation of the water at the Screen House must either be measured, or calculated by the methods already described, and the corresponding level of the tailrace observed, or calculated from the diagrams of the tailrace gauge relations. The difference between these elevations gives the effective head to be applied in the above-mentioned equation.

From the available data we have prepared a table of the mean monthly electrical horse-power output available at the site from 1860 to 1907, and a diagram showing this output is included as page D-53. The table is as follows:

Table of Mean Monthly Power Output Available
1860 - 1907

Month	Gauge Elevations - Mean Monthly Readings: Cleveland U.S.L.S. Datum	Oswego U.S.L.S. Datum	Chippawa H.E.P.C. Datum	Screen-House H.E.P.C. Datum	Tailrace H.E.P.C. Datum	Effective Head Feet	Flow c.f.s.	Electrical Horse-power Available
Jan.	572.10	245.58	560.52	544.75	246.21	298.54	18,200	554,700
Feb.	572.03	245.55	560.49	544.58	246.31	298.27	18,200	554,200
Mar.	572.22	245.91	560.60	545.20	246.67	298.53	18,200	554,700

Table of Mean Monthly Power Output Available
1860 - 1907 (Continued)

Month	Gauge Elevations - Mean Monthly Readings: Cleveland U.S.L.S. Datum	Oswego U.S.L.S. Datum	Chippawa H.E.P.C. Datum	Screen House H.E.P.C. Datum	Tailrace H.E.P.C. Datum	Effective Head Feet	Flow c.f.s.	Electrical Horse-power Available
Apr.	572.70	246.49	560.87	546.42	247.39	299.03	18,200	555,500
May	573.05	246.82	561.08	547.18	247.79	299.39	18,200	556,200
June	573.24	246.95	561.18	547.47	247.95	299.52	18,200	556,500
July	573.22	246.93	561.15	547.35	247.93	299.42	18,200	556,300
Aug.	573.04	246.63	561.06	547.10	247.58	299.52	18,200	556,500
Sept.	572.78	246.22	560.90	546.56	247.08	299.48	18,200	556,300
Oct.	572.45	245.88	560.73	546.10	246.53	299.47	18,200	556,300
Nov.	572.21	245.63	560.60	545.18	246.28	298.90	18,200	555,300
Dec.	572.14	245.57	560.55	544.88	246.18	298.70	18,200	555,000

It will be noted that the power available is lower in the months of December, January, February and March, with the minimum in February. During these four months also the presence of ice in the upper river may necessitate the use of the submerged gathering tubes for part of the time. With the tubes in operation the level at Chippawa will probably be lowered by about 0.5 feet, producing a reduction in the power output not shown in the table. This decrease in level at Chippawa would react on the level at the screen house, lowering the level at that point by five or six times the decrease at Chippawa, and resulting in a net decrease in head of about three feet, making the output about one per cent. less for the same flow.

A study has been made of the records available from 1910 to 1916 inclusive to determine the average number of days in each year when heavy, broken ice may be expected in the upper river. It would appear from the figures that it would be necessary to use the gathering tubes for an average of about 28 days per year

HYDRO-ELECTRIC INQUIRY COMMISSION

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QUEENSTON-CHIPPAWA POWER DEVELOPMENT

AVERAGE MONTHLY POWER OUTPUT AVAILABLE 1860 TO 1907

Toronto, July 30th, 1923. Made by G.&B. Checked by W.J.F.

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when the full capacity of the canal is being used. The greatest demand for the protection afforded by the submerged intake would occur in April during the spring break-up, and the next greatest in January and February while ice is forming in the river. The figures for 1910 to 1916 inclusive are shown in the following table:

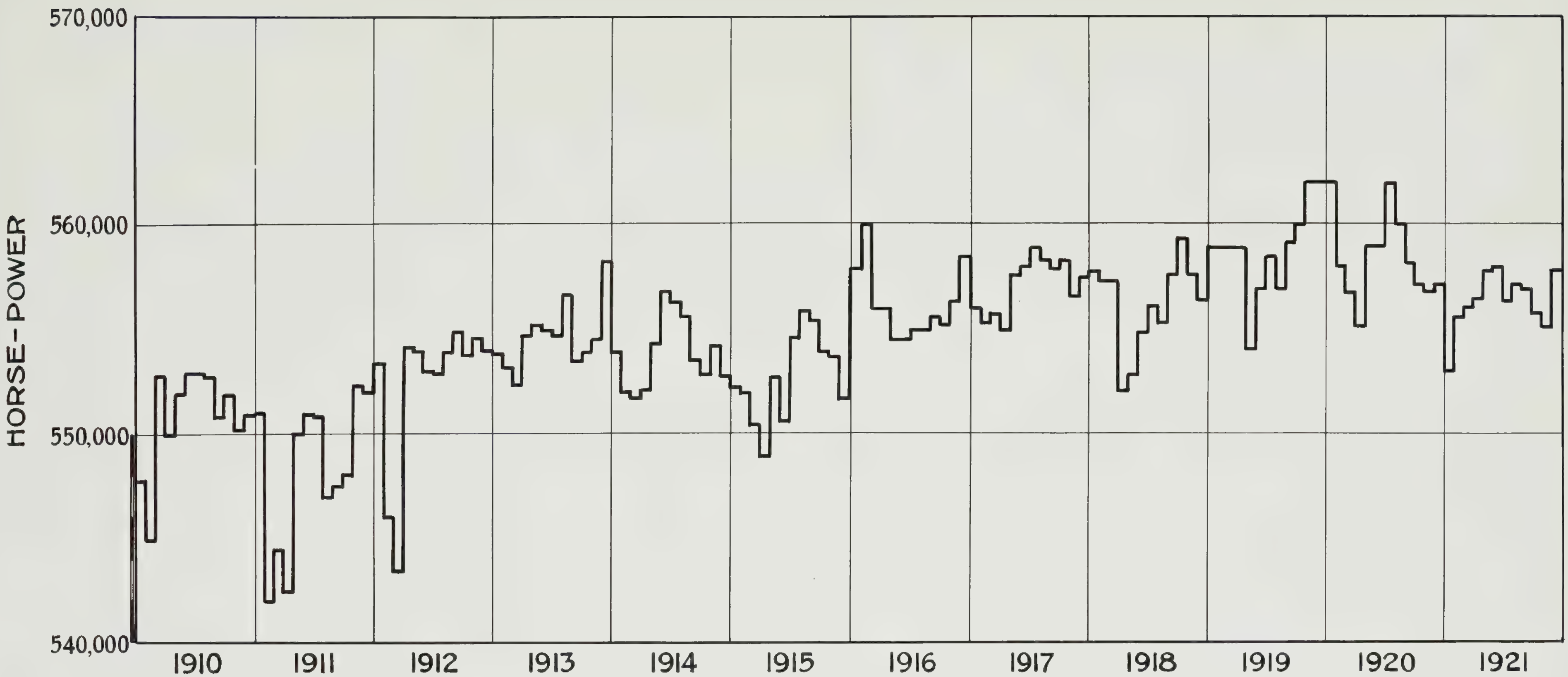
Table of Days of Flow of Field Ice

Month	Year						
	1910	1911	1912	1913	1914	1915	1916
December	6	0	0	0	7	0	2
January	7½	6½	14	0	2	6	6½
February	6½	4½	10	8	7	2	13
March	10	2	4	1	0	6	4
April	10½	5	3	6	6	8	22
Total	40½	18	31	15	22	22	47½

Mean = 28 days per year.

In order to study further the power output available, a diagram has been prepared to show the mean monthly power output of the Queenston-Chippawa plant on the assumption that it was in operation from the period from 1910 to 1921 inclusive. This diagram is shown on page D-55. Another diagram, included as page D-56, shows a replotting of the power output available from 1910 to 1916 inclusive, using the synchronous stages of Lake Erie and of Lake Ontario. These diagrams also show that the period of minimum power available is in the winter. The last-mentioned diagram also shows that the critical drop in the power is

(D-55)



HYDRO-ELECTRIC INQUIRY COMMISSION

W. D. GREGORY, CHAIRMAN

QUEENSTON-CHIPPAWA POWER DEVELOPMENT

AVERAGE MONTHLY POWER OUTPUT AVAILABLE 1910 TO 1921

Toronto, July 30th, 1923. Made by G.E.B. Checked by W.J.F.

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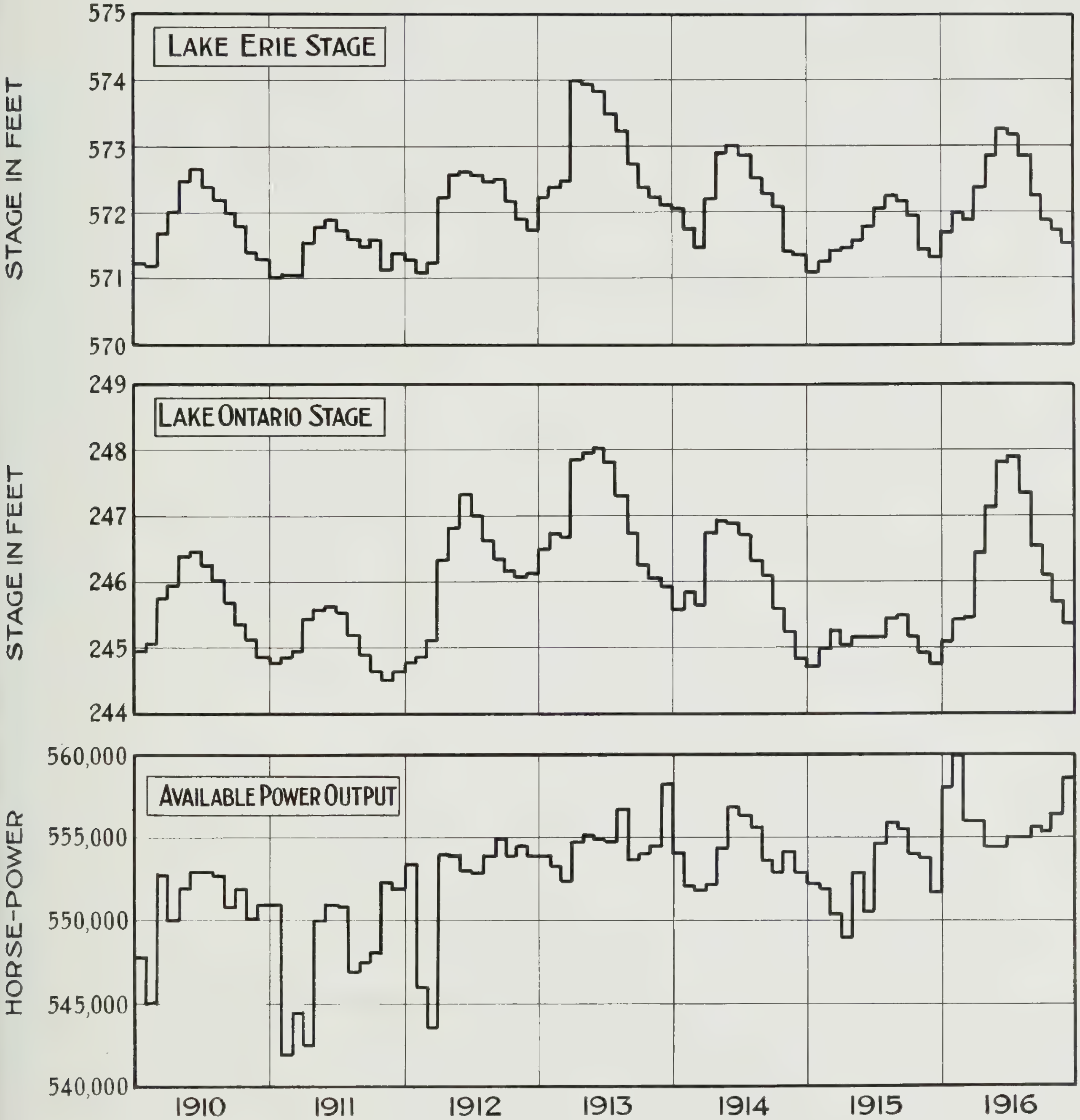

HYDRO-ELECTRIC INQUIRY COMMISSION

W. D. GREGORY, CHAIRMAN

QUEENSTON-CHIPPAWA POWER DEVELOPMENT

RELATION OF AVAILABLE POWER OUTPUT TO STAGE IN LAKE ERIE AND LAKE ONTARIO

Toronto, July 30th, 1923. Made by S.G.B. Checked by W.D.G.

WALTER J. FRANCIS & COMPANY

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caused by the stage at Chippawa being so low that to preserve the surface level at the Screen House at Elevation 542.0 the discharge would have to be reduced below 18,200 cubic feet per second. It should be noted in this connection that while the stages of the two lakes vary almost simultaneously, the stage at the Screen House varies much more quickly than that at Chippawa; therefore, when the stage at Lake Erie is high, the stage at the Screen House is relatively higher than when the Lake Erie stage is low. The result is that the effective head on the plant at Queenston is greater during high stages of the lakes than during low stages, even if the relative difference between the lakes were the same.

In detail analysis, the possible performance of the plant under extremely low water conditions has been studied, and the mean monthly power output available has been plotted for 1895 and 1896, the two lowest years on record. This diagram, included as page D-58, shows a total variation of about two per cent. in the power output. The minimum mean monthly power output available would have been 544,000 horse-power in March of 1896, without allowing for loss of head by the possible use of the gathering tubes at the intake.

It is not unreasonable to assume that due to peculiar and very occasional combinations of certain meteorological conditions, a brief change in the water stage at Chippawa might occur and obtain for a few hours, or at most a day, during which the level might be a foot or a foot-and-a-half below the monthly mean. Should such a condition be combined with ice conditions demanding the use of the gathering tubes the loss of head might possibly be such that the

HYDRO-ELECTRIC INQUIRY COMMISSION

W. D. GREGORY, CHAIRMAN

QUEENSTON-CHIPPAWA POWER DEVELOPMENT

AVERAGE MONTHLY POWER OUTPUT AVAILABLE, 1895 AND 1896

Toronto, July 30th, 1923. Made by S.R.W., Checked by W.J.F.

WALTER J. FRANCIS & COMPANY

CONSULTING ENGINEERS

available power output for the day might be as low as 500,000 horse-power. This is on the assumption that the Screen House level would not be below Elevation 542.00 being the level of the bottom of the curtain wall at the Screen House.

A factor which will assist in raising and maintaining the power output of the plant is the daily pondage of reservoir capacity of the Welland River and the Canal. The total surface area of the available pond formed by the lower stretch of the Welland River and the Canal is on the order of ten or twelve million square feet or more. A draft of one foot over this area represents a flow of about 1,000 cubic feet per second over a period of three or four hours, equivalent to an increased power output of 30,000 horse-power or more. These figures are approximate only, and it is conceivable that the pondage extending as it does many miles up the Welland River, might yield as much as 50,000 horse-power for several hours.

With a flow of 18,200 cubic feet per second it would appear that the effective head available would be usually about 299 feet. By drawing down the Canal and by allowing a level in the Forebay of about Elevation 540.00, it is likely that the effective head for a peak flow of, say, 19,000 cubic feet per second would not be less than 292 feet, representing a power output of about 567,000 horse-power.

It would therefore appear reasonable to complete the plant for ten main units of about the capacity of the present machines. Under ordinary conditions, the more recently installed units are capable of delivering at high efficiency more than their rated capacity. Nine units having characteristics similar to

those of Units Nos. 4 and 5 would probably deliver 500,000 horse-power, or possibly 550,000 horse-power, while ten units could be called upon in low water periods, at a higher efficiency. The tenth unit, while normally considered a spare unit in accordance with standard practice for a plant of more than five or six units, would doubtless be used for a large part of the time with the other nine to increase the normal output at a better efficiency of the whole plant.

Summing up the available data it would appear that 550,000 electrical horse-power delivered to the 12,000-volt bus bars, under practically all conditions of operation, may be considered as the commercial capacity of the Queenston-Chippawa Power Development.

COPY

Test of Unit No. 5.

The Power House.

At the present time half of the power house is completed and five of the main units are in running order. Construction work is in progress for the superstructure for Unit No. 6 and for the substructure of Units Nos. 7 and 8. The general condition of the plant is shown clearly in the photograph included herewith as page D-61 which was taken on April 5th, 1923, while Units Nos. 1, 2, 3 and 4 were in operation. A general proof of the efficiency of the units is the comparative lack of motion in the tailrace water, as it is practically impossible to determine the number of units in operation from an examination of

To face page D-61.

No. 5

Test of Turbine No. 5, May 20th, 1923.

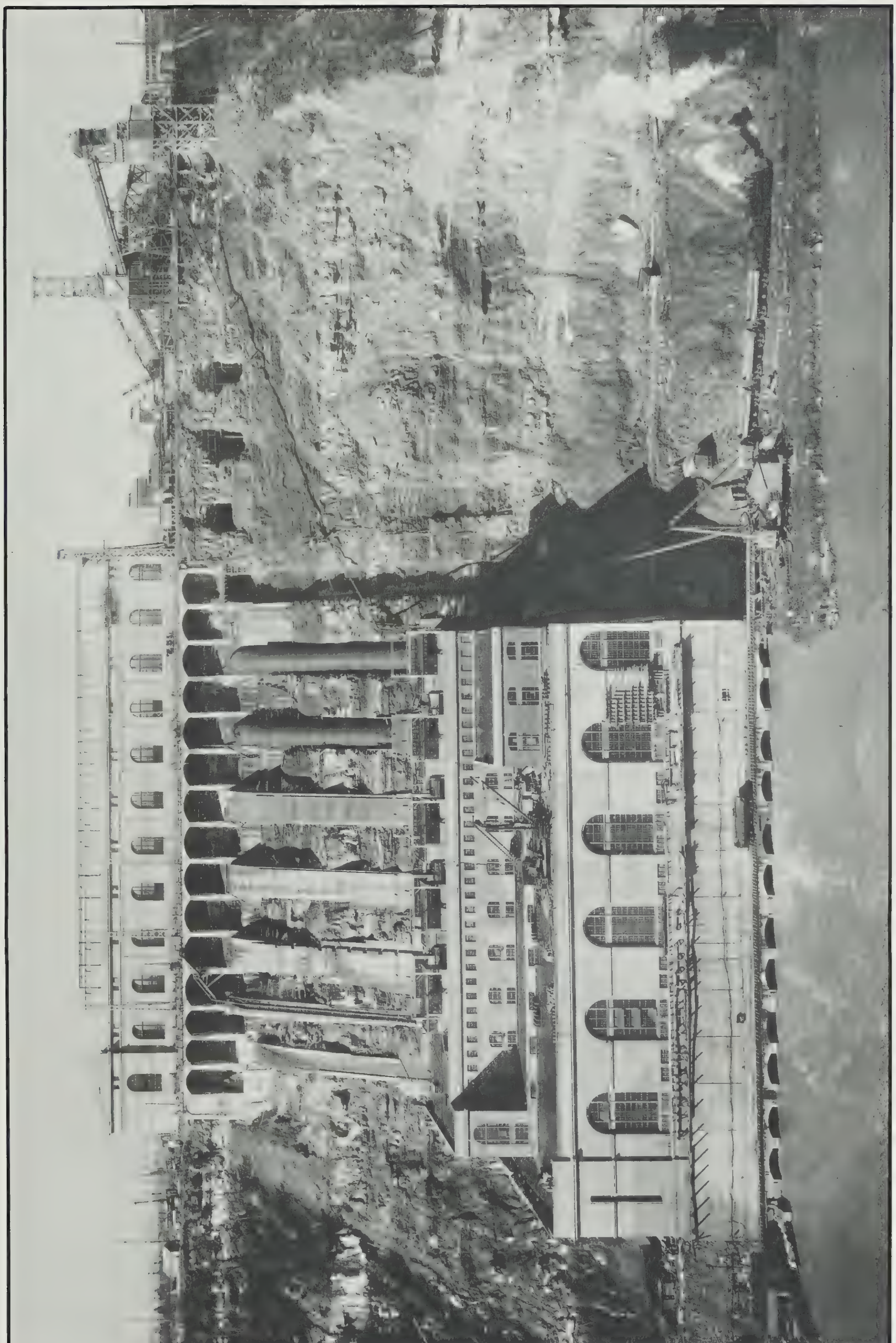
Photograph showing

General View of Power House, Penstocks and Screen House

looking westerly from the American side of the
Niagara River,

Taken April 5th, 1923.

Note: When the above photograph was
taken Generators 1, 2, 3 and 4
were all under load.



the tail water in the photograph.

The first of the main units was put into operation at the beginning of the year 1922, and since then four other units have been put into commission successively. The curves shown on the diagram included herewith as page D-63 give the daily output of the plant on five separate days chosen at random. The curves all have the same characteristics, and they may be considered as representative of the general daily output of the plant at present, except for Tuesdays when the load is greatest, and for Sundays when the load is least.

Test of Turbine No. 5.

COPY

All of the main units have been regularly tested, and all except No. 1 have shown practically the same results. The slightly lower efficiency of Unit No. 1 is undoubtedly due to the fact that it has an ordinary curved draft tube and not a Moody spreading draft tube like the others. On May 20th an official test was made of Turbine No. 5 to determine its efficiency and capacity and the other hydraulic characteristics of the unit. The results of this test have been used in the discussions of commercial capacity of the Queenston-Chippawa Power Development.

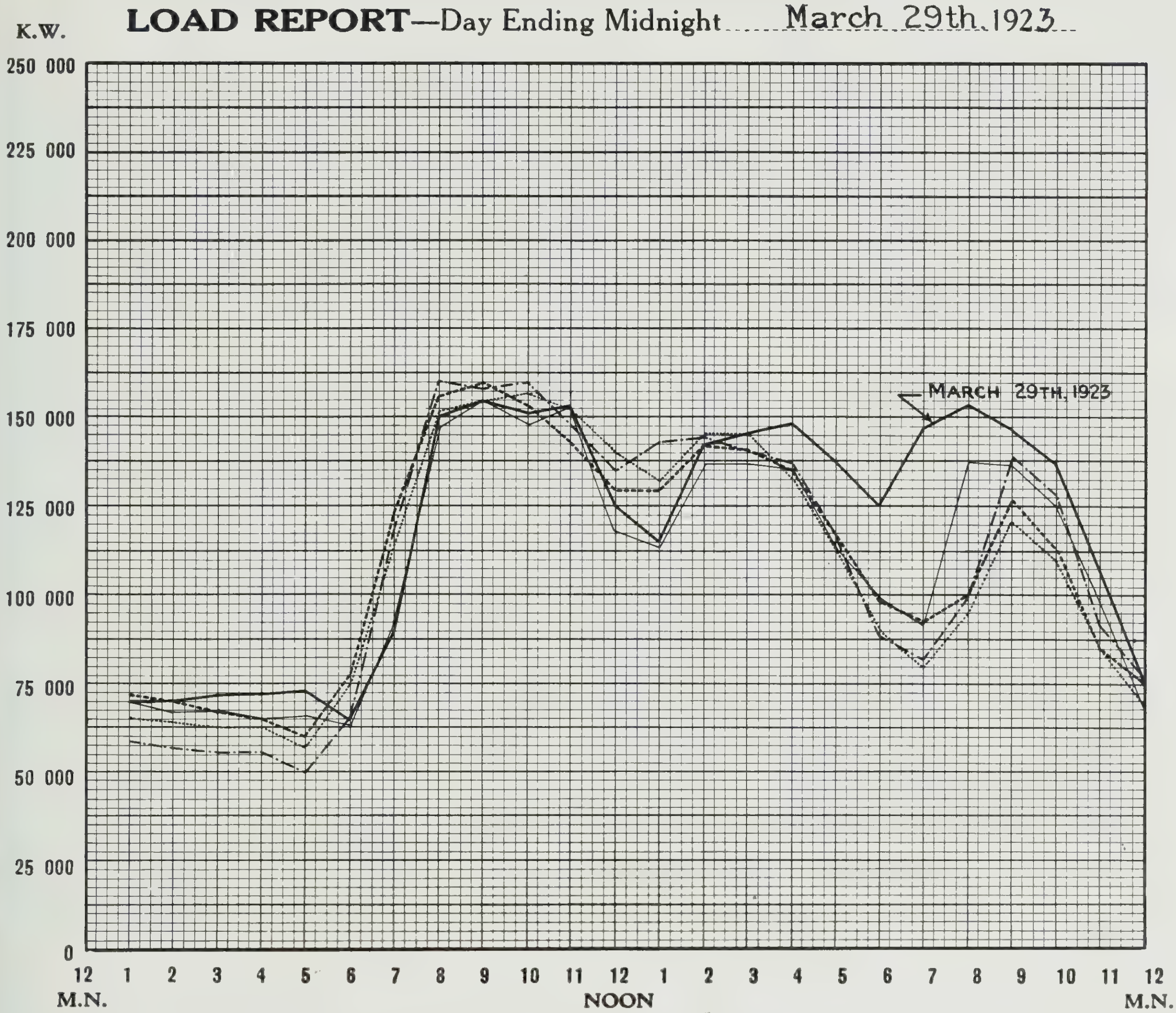
The preparations were made during the days immediately preceding the observations and were conducted under the immediate direction of Mr. Acres and Mr. Hogg. Mr. Francis, accompanied by members of his staff, was present throughout the test and witnessed all the operations.

A general cross section of Unit No. 5, showing the principal features, is included as page D-65. The diagram included herewith as page D-66 shows a

HYDRO-ELECTRIC POWER COMMISSION OF ONTARIO

QUEENSTON GENERATING STATION

LOAD REPORT—Day Ending Midnight March 29th 1923



FOUR OTHER DATES ARE SHOWN AS FOLLOWS:

APRIL 26TH, 1923 ———————
MAY 23RD, 1923 -·-·-·-·-·-·-
JUNE 28TH, 1923 - - - - - - - - -
JULY 26TH, 1923 ·····················

SUMMARY OF STATION RECORDS

GENERATION BY UNITS	
UNIT	KILOWATT HOURS
No. 1	730,000
No. 2	860,000
No. 3	650,000
No. 4	730,000
No. 5	
Total	2,970,000

TOTAL GENERATION		
	K. W.	TIME
Maximum 1-Minute Peak	158,000	8·23 A.M.
" 20 " "	155,000	11·19 A.M.
" 60 " "	151,000	8·06 A.M.
Total Kilowatt Hours	2,970,000	
Average Kilowatts	123,800	
Load Factor	78·4%	

(Load Factor = Ave. K.W. ÷ 1-Min. Peak)

Service **A**	K. W. H.
Service **B**	K. W. H.

Station Capacity K. W.

General Cross Section of Unit No. 5, together with the Location of the Observation Stations. In the Screen House there was a staff gauge and a head-water float gauge to determine the elevations and variations in water level. At about Elevation 350 an observation station was located to determine the pressure in the penstock by means of a Gibson instrument. Midway between the Johnson valve and the turbine casing a mercury manometer was connected to the penstock, and at the tailrace there was a float gauge to measure the tail water levels during the test. On the governor a special hand-control was installed to actuate the pilot valve. A servomotor indicator was located at the gate opening apparatus to determine a record of the gate opening. Special electrical measuring instruments were installed in the control room to measure the electrical output. All the observation stations were inter-connected by telephones and signal systems so that there was immediate inter-communication between the observers.

All orders were given by Adolf Aeberli, M.E., Mechanical Engineer of the Hydraulic Department, who was stationed at the governor and control stand in the generator room. A. A. McLaren, Chief Inspector of Power House Construction, was in charge of the surge gauge in the Screen House. H.W. Wagner, B.A.Sc., Assistant Field Engineer, was in charge of the staff gauge in the Screen House. E. Pickles, Designing Draftsman, was in charge of the Gibson Instrument at Elevation 350. J. J. Traill, C. E., Engineer of Tests, Hydraulic Department, was in charge of the manometer readings and the preparation of the results. A.W. McQueen, Assistant Engineer, Hydraulic Department, was in charge of the servomotor indicator. C. H. Ellis, Junior Instrument Man, assisted Mr. Traill.

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COPY FOR ENCLOSURE TO Mr. J. Allan Ross.

To face page D-65.

No. 6

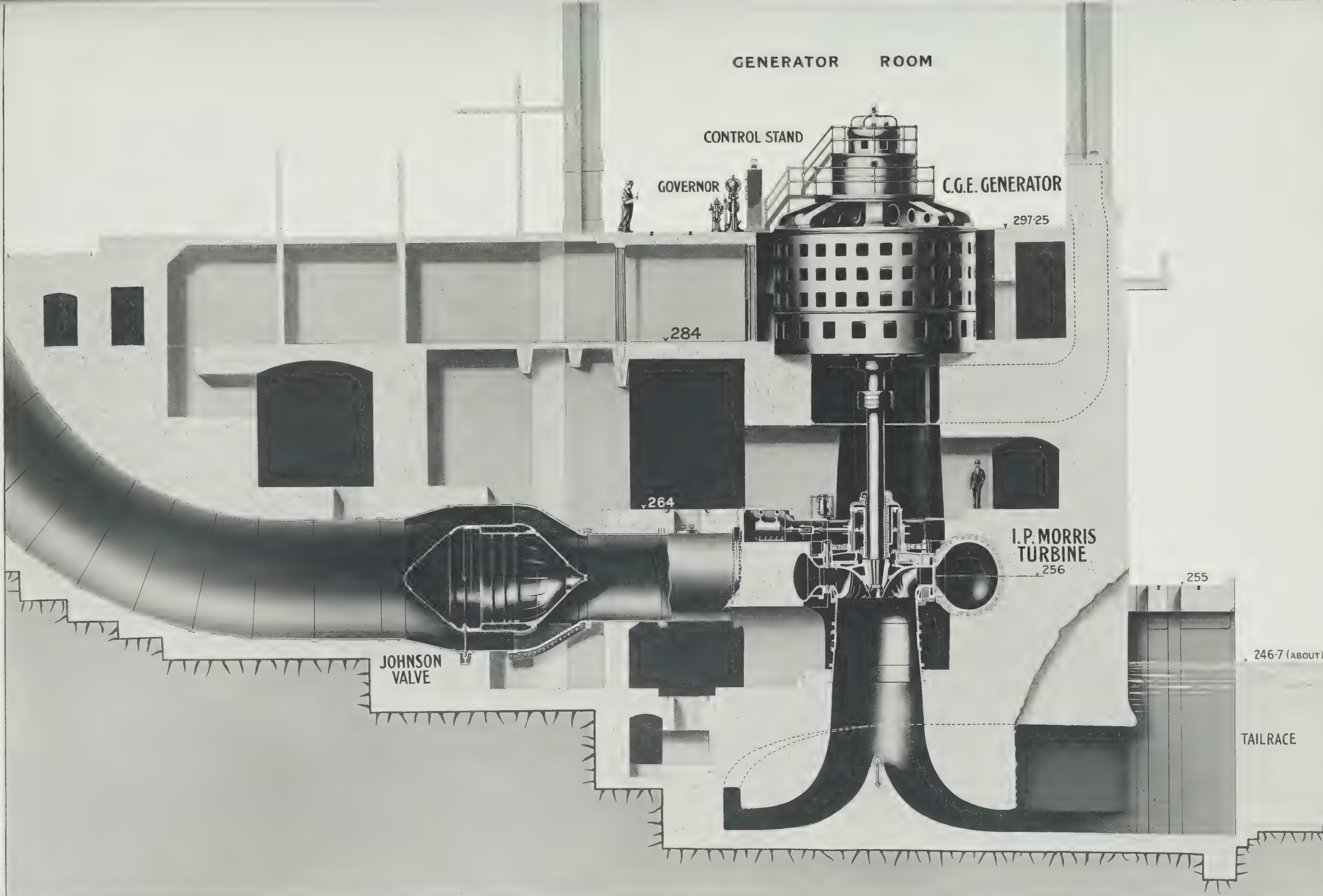
Test of Turbine No. 5, May 20th, 1923.

Photograph of Wash Drawing
showing

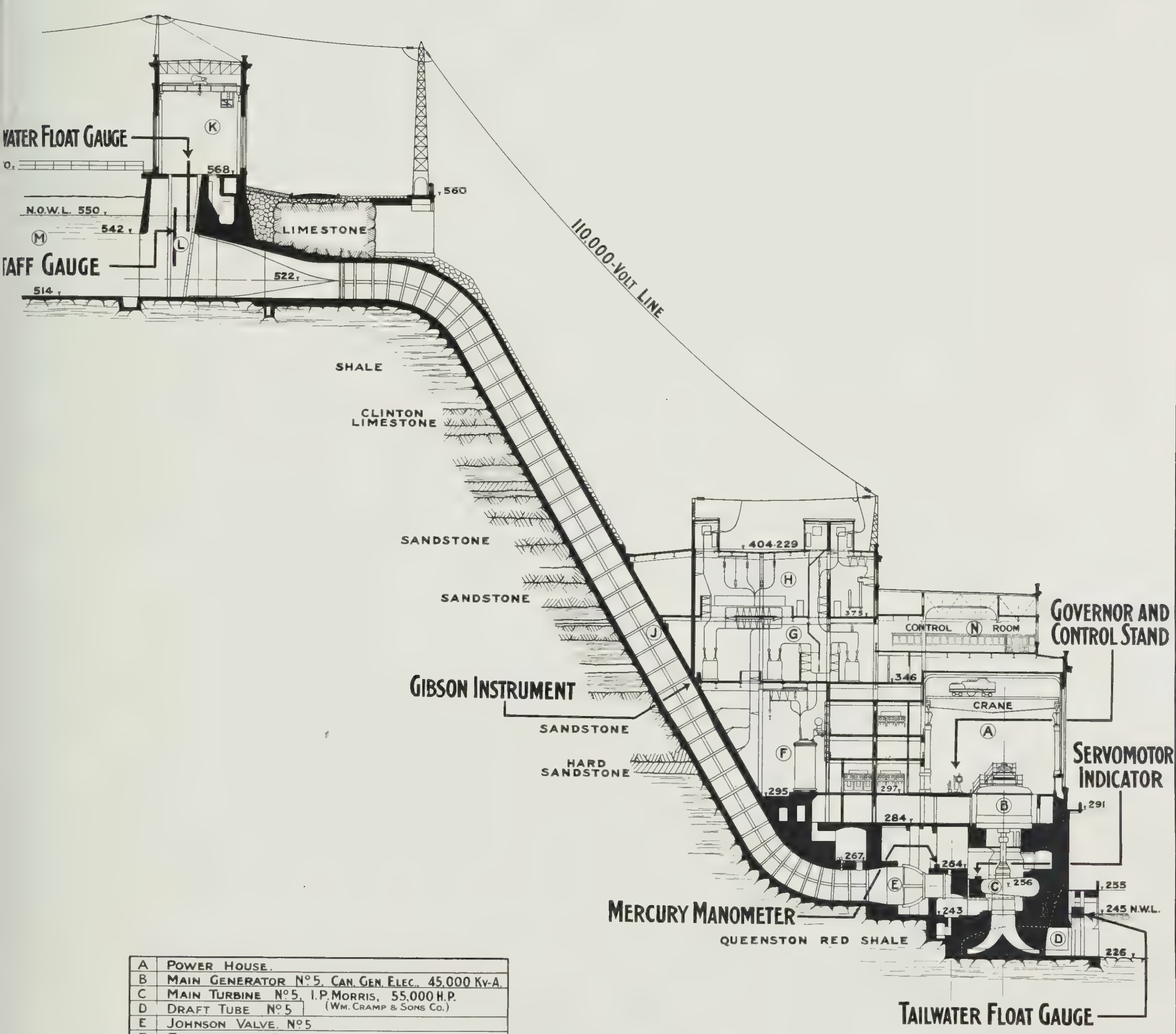
General Cross Section at Centre Line of Turbine No. 5.

Made July 27th, 1923.

Note: This photograph, illustrating the principal features of the installation below the general level of the floor of the generator room, was made from a scale drawing in which was combined a wash drawing made by Messrs. Wm. Cramp and Sons Ship and Engine Building Company, I. P. Morris Department, to show the hydraulic machinery.


GENERATOR ROOM
CONTROL STAND
GOVERNOR
C.G.E. GENERATOR
297·25
284
264
I.P. MORRIS
TURBINE
256
255
246·7 (ABOUT)
TAILRACE
JOHNSON
VALVE

(D-65)



A	Power House.
B	Main Generator N°5, Can. Gen. Elec., 45,000 Kv-A.
C	Main Turbine N°5, I.P. Morris, 55,000 H.P.
D	Draft Tube N°5 (Wm. Cramp & Sons Co.)
E	Johnson Valve. N°5
F	Transformer Room.
G	High Tension Switches
H	High Tension Bus Bars.
J	Penstock N°5
K	Screen House.
L	Screen N°5
M	Forebay
N	Control Room

Elevations Referred to H.E.P.C. Datum shown thus – 291

Scale of Feet

Hydro-Electric Inquiry Commission

W. D. Gregory, Chairman

Queenston-Chippawa Power Development

Test of Turbine N° 5, May 20th., 1923

Section Showing Observation Stations

Toronto, July 30th., 1923, Made by S.R.W., Checked by W.J.F.

Walter J. Francis & Company

Consulting Engineers

L.W. May, Junior Instrument Man, was in charge of the tail water gauge. G.D. Floyd, Assistant Laboratory Engineer of the Electrical Department, H.S. Baker, Meter Supervisor of The Ontario Power Company, and J. I. Cram, B.A.Sc., Meter Engineer of The Ontario Power Company, were in charge of the electrical readings. J. Rapelie, Chief Operator of the power house of The Ontario Power Company, operated the governor.

S. L. Kerr, B. Sc. in M. E., was also present throughout the test, representing the manufacturers of the turbine and the governor, making his own observations and diagrams as the test proceeded.

A comprehensive idea of the observation instruments may be gained by reference to page D-68 for the Gibson surge gauge in the Screen House, to page D-69 for the Gibson penstock pressure instrument at Elevation 350, to page D-70 for the mercury manometer connected to the penstock at the entrance to turbine casing, to page D-71 for the tailrace gauge, to page D-72 for the hand-control on the governor, to page D-73 for servomotor indicator and to page D-74 for the electrical test instruments in the control room.

Procedure in the Test.

The measurement of water was made by the Gibson method. In carrying out a test by the Gibson method the unit to be tested is brought up to speed and synchronized with the other units in the plant, and load is added until the desired amount of load is reached. On signal from the observer at the Gibson penstock pressure instrument, which is the principal Gibson instrument and that to which reference is made in speaking of "the Gibson instrument" and

[illegible] Kay, Junior Instrument Man, was in charge of the [illegible]

[illegible], Assistant Laboratory [illegible]

[illegible], [illegible] of the [illegible] Power Company, [illegible]

[illegible], [illegible] of the Ontario Power Company, [illegible]

electrical readings. [illegible] of the power house of the Ontario Power Company, operated the governor.

[illegible] was also present throughout the test, representing the manufacturers of the turbine and the governor, [illegible]

A comprehensive idea of the observation instruments may be gained by reference to page D-[illegible] for the [illegible], to page D-[illegible] for the Gibson generator pressure instrument at Elevation [illegible], to page D-[illegible] for the [illegible], to page D-[illegible] for the [illegible], to page D-[illegible] for the [illegible] on the governor, to page D-[illegible] for [illegible], to page D-[illegible] for the electrical test instruments in the control room.

Procedure in the Test.

The measurement of water was made by the Gibson method. In a typical run a test by the Gibson method the unit to be tested is brought up to speed and synchronized with the other units in the plant, and load is added until the desired amount of load is reached. On signal from the observer at the Gibson generator pressure instrument, which is the principal Gibson instrument and that to which reference is made in speaking of "the Gibson instrument" and

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To face page D-68.

No. 7

Test of Turbine No. 5, May 20th, 1923.

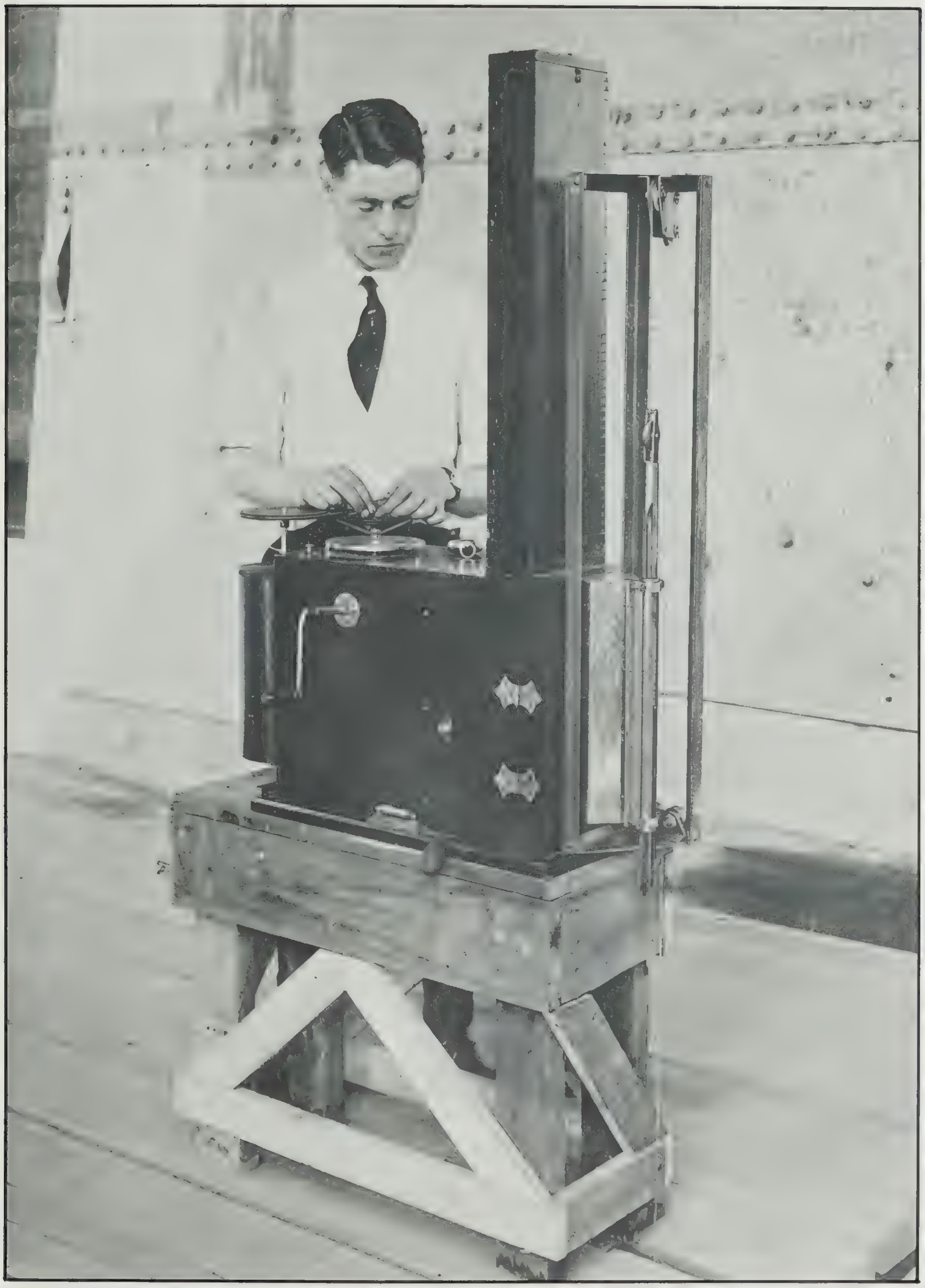
COPY

Photograph showing

Surge Gauge in Screen House with Gibson Instrument attached.

Taken May 19th, 1923.

Note: The cord passes through the floor to a float on the surface of the water. The movement of the float causes a corresponding motion of the plunger attached to the front of the Gibson Instrument. In the plunger is a narrow slot, the movement of which makes an exposure on the film within the Gibson Instrument.



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To face page D-69.

No. 8

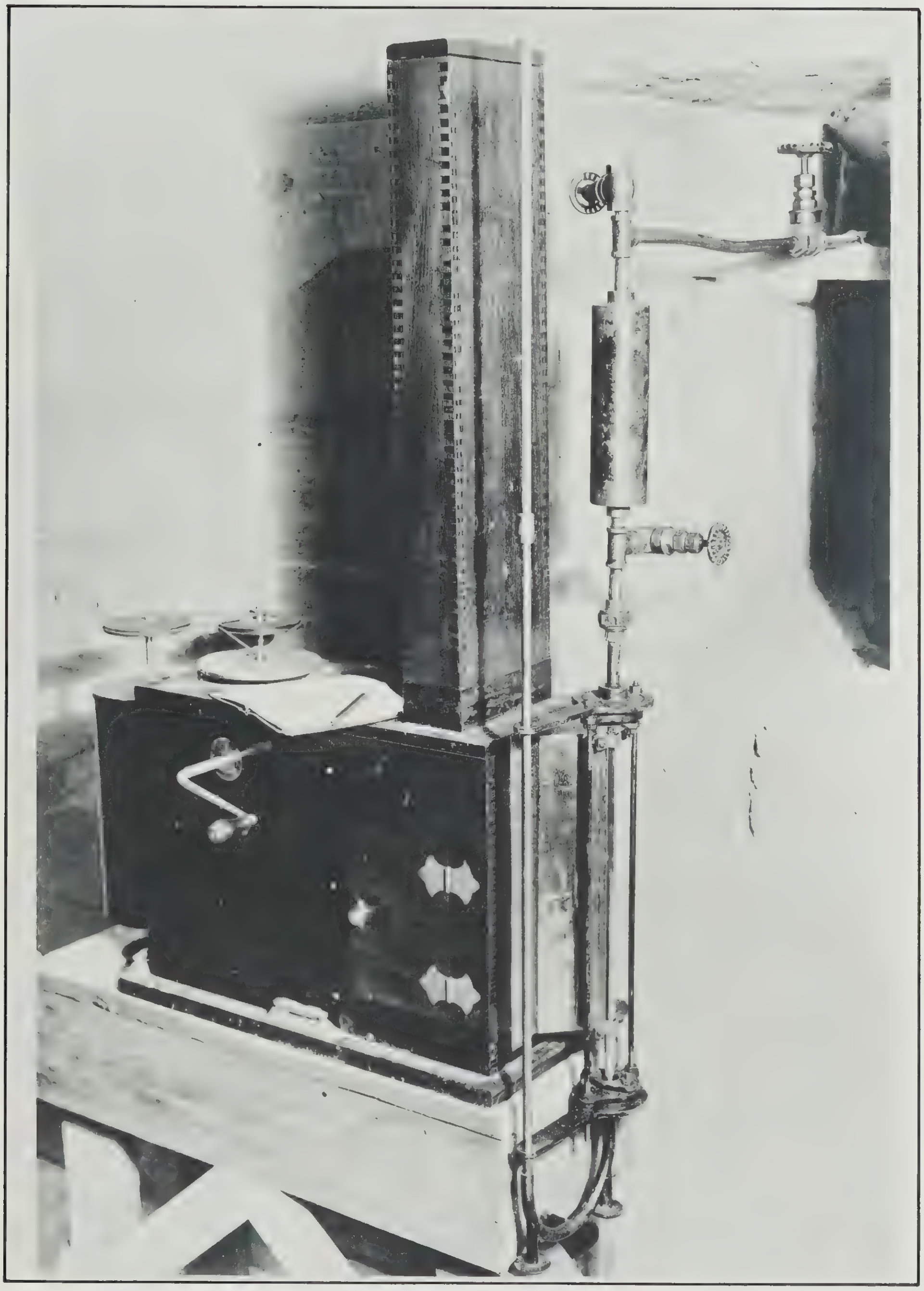
Test of Turbine No. 5, May 20th, 1923.

Photograph showing

Gibson Instrument measuring Penstock Pressure at Elevation 350.

Taken May 19th, 1923.

Note: The piping and valves at the upper right hand side of the picture are connected to the penstock. The riser pipe in the centre of the picture extends about fifteen feet above the Gibson Instrument.



To face page D-70.

No. 9

Test of Turbine No. 5, May 20th, 1923.

COPY

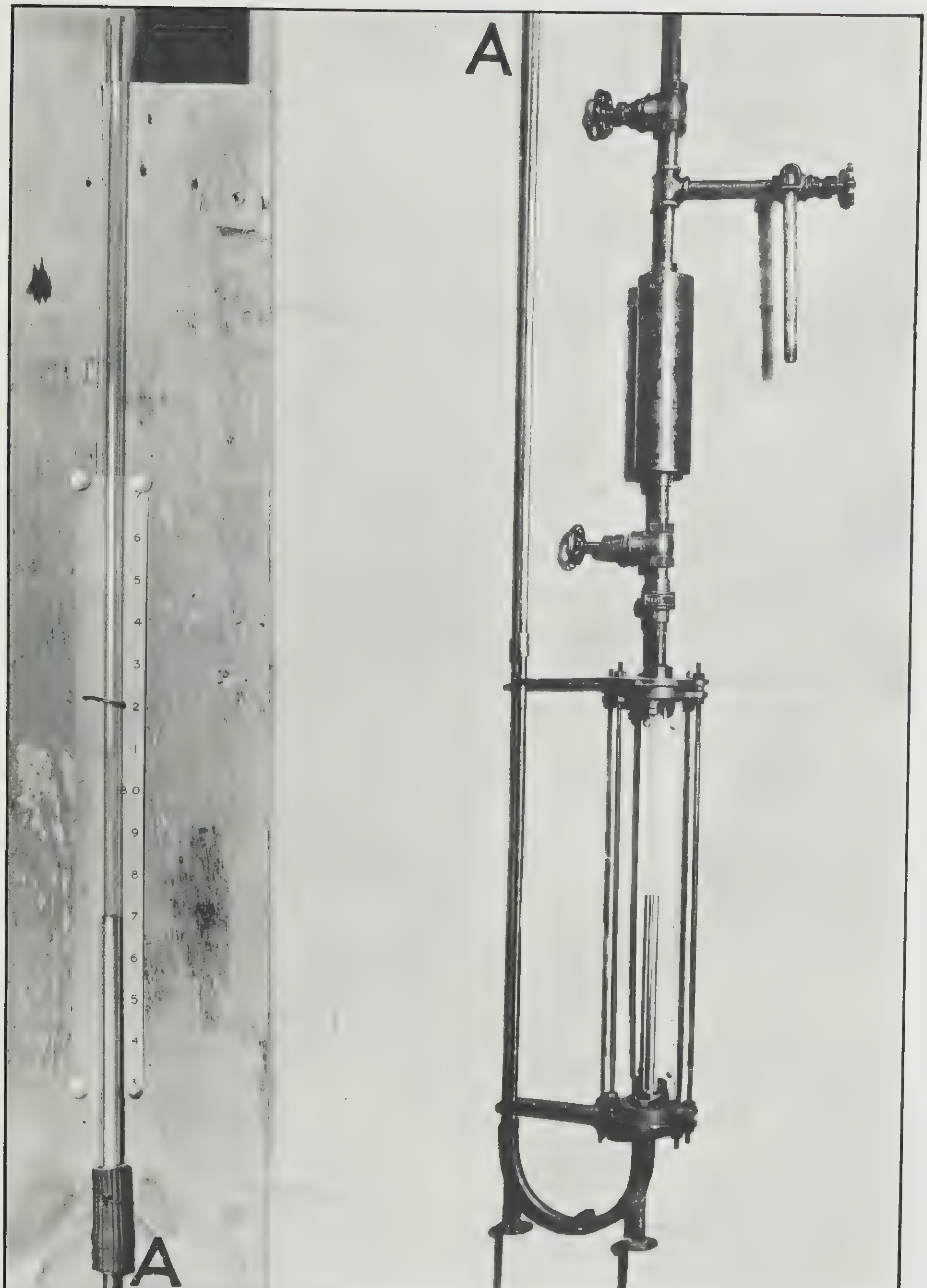
Photograph showing

Mercury Manometer connected to Penstock at Entrance to Turbine Casing.

Taken May 19th, 1923.

Note: The glass tube on the right is connected to the brass tube near the centre of the picture so that the two points marked A coincide.

The 7-foot mark on the right-hand scale is at Elevation 267. The 8-foot mark on the left-hand scale is at Elevation 288.


A
A

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To face page D-71.

No. 10

Test of Turbine No. 5, May 20th, 1923.

COPY

Photograph showing

Float Gauge Indicator for Tail Water Elevations.

Taken May 19th, 1923.


4
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To face page D-72.

No. 11

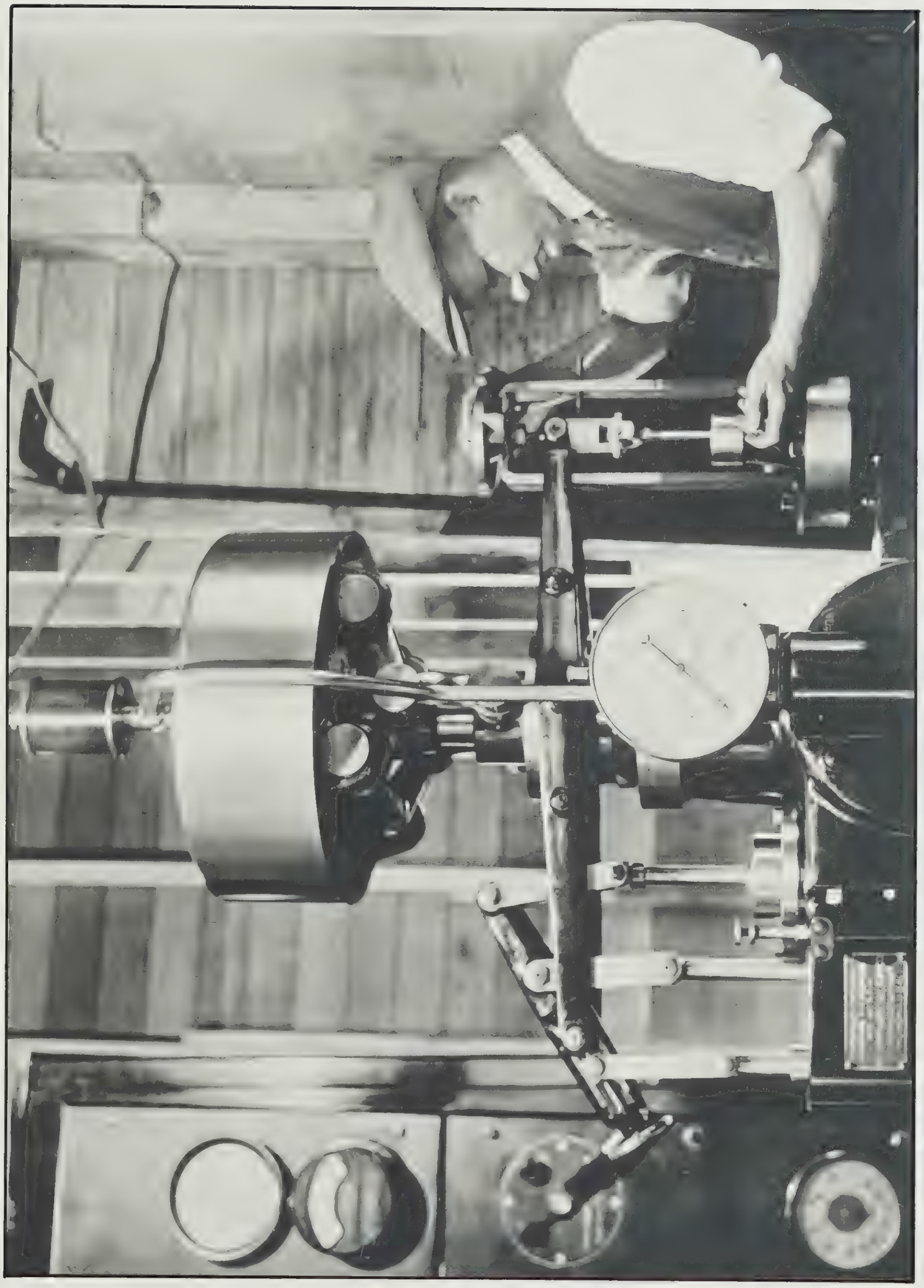
<u>Test of Turbine No. 5, May 20th, 1923.</u>

COPY

Photograph showing

<u>Governor Head at Control Pedestal showing Special Hand-control used to move the Pilot Valve during the Test.</u>

Taken May 19th, 1923.



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To face page D-73.

No. 12

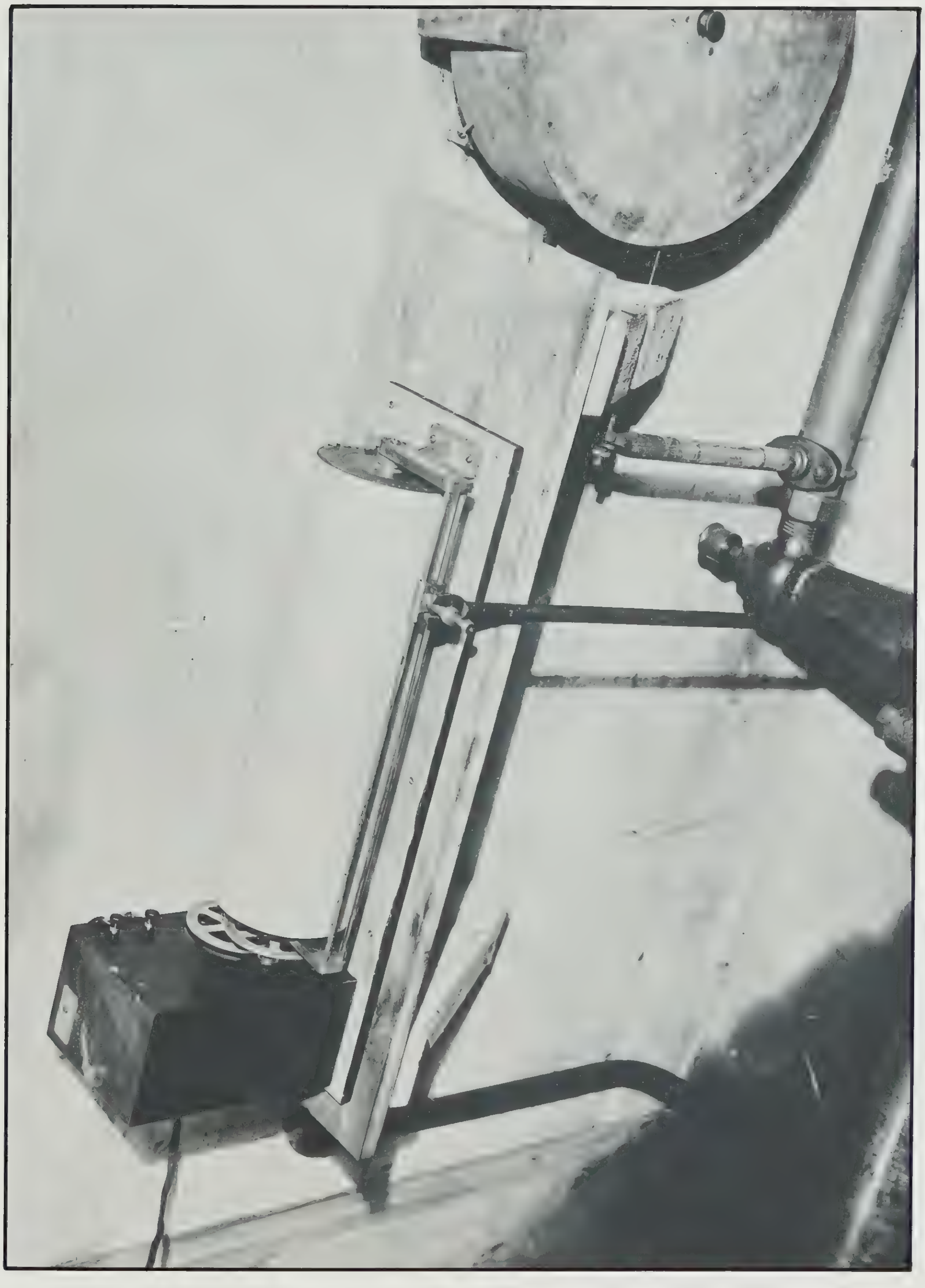
Test of Turbine No. 5, May 20th, 1923.

COPY

Photograph showing

Servomotor Indicator for Measuring the Gate Opening during the Test and the Time of Gate Closure.

Taken May 19th, 1923.



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To face page D-74.

No. 13

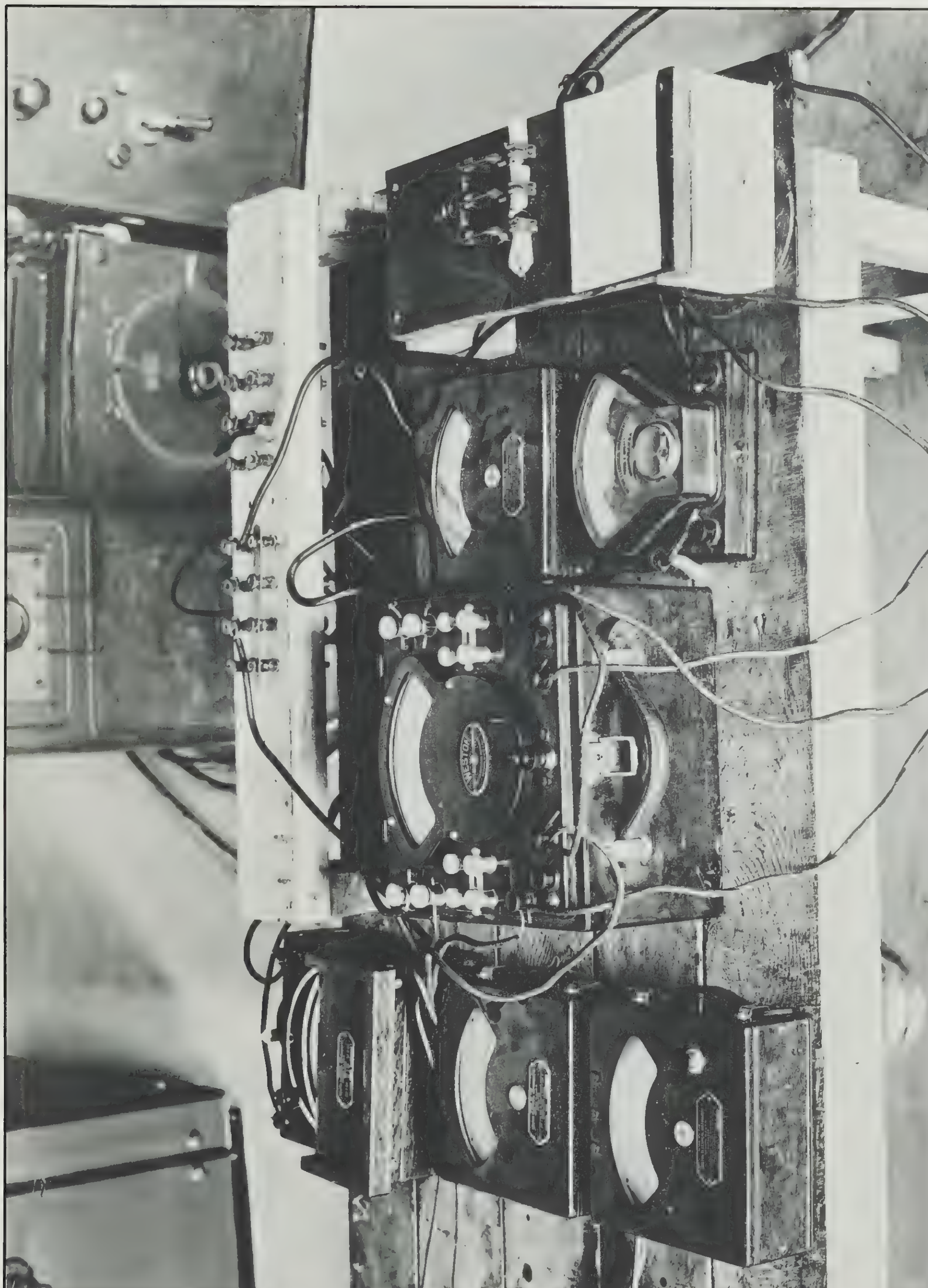
Test of Turbine No. 5. May 20th, 1923.

COPY

Photograph showing

Electrical Meters set up temporarily in Control Room.

Taken May 19th, 1923.



"the Gibson diagram" in the succeeding text, the operator at the turbine governor moves the pilot valve so as to cause the governor to close the turbine gates completely. The unit remains in step with the other units in the plant, but the flow of water to it is shut off entirely, and its load is picked up by the other units. Stoppage of the flow of water causes a rise in pressure in the penstock, a graph of which, with respect to time, is made by the Gibson instrument. From this graph is determined the amount of water flowing in the penstock at the instant at which the turbine gates began to close. Observations of head and power are made also at the instant at which the shut down of the turbine gates commences.

Head Water. The staff gauge in the Screen House was read immediately before shut down, during shut down when the upward surge of the water surface reached its maximum, and about two minutes later when the surge in the Screen House had died out. The float gauge with a Gibson instrument attached to facilitate recording was used to draw a graph of the variations in water level in the Screen House during the period of shut down and for a few seconds thereafter, covering in all a period of thirty or forty seconds. This graph is used to correct the Gibson diagram on the penstock pressure instrument.

Turbine Casing Pressures. The mercury manometer, connected to the penstock about midway of the length of the ten-foot pipe between the Johnson valve and the turbine casing, is joined thereto by four connections equally spaced on the circumference of the penstock. The manometer is about twenty-three feet in height, with a short length of glass tubing on the pressure side

at the lower end, and on the atmospheric side at the upper end which is open to the air. The relations to elevations are given on the facing page of the photograph of the manometer on page D-70.

Readings were taken of the level of the mercury column in each of the glass tubes immediately before load was thrown off the unit, and again about two minutes later when the oscillation of the mercury column had ceased and the surges had died out in the Screen House. This latter reading was coincident with the third reading taken on the staff gauge in the Screen House, and also with what is known as the final static reading on the Gibson diagram. The final reading of the manometer is used to measure the specific gravity of the mercury and to check the first readings in each test.

Tail Water. The float gauge measuring tail water levels was situated in a stilling box in one of the stop-log checks in the central pier of the tail bay of Unit No. 5. Readings were taken on this gauge at intervals of fifteen seconds for a period of about two minutes, starting about one minute before the load was thrown off the unit.

Penstock Measurement. The Gibson instrument for measuring the penstock pressure and the discharge of the unit was set up and calibrated on May 19th, 1923, and remained undisturbed after the calibration until the test commenced. The surge gauge in the Screen House provided the information with which to correct the Gibson diagram in regard to the variations in the head water level during the period in which load was thrown off the unit. The two diagrams were synchronized by marking the surge diagram on signal from the Gibson instrument at the instant at which the shut down commenced.

As the Gibson diagram records only the amount of water shut off in the penstock, and as, with the turbine gates tightly shut, there is still a small leakage through the gates, it was necessary to make a leakage test with the gates closed. This leakage test was carried through on the morning of May 19th, by closing the head-gates in the Screen House and observing the time the water surface in the penstock subsided from level to level. After a period of about an hour the Johnson valve in the penstock was closed, and the time required for the water surface to rise as a result of the leakage through the head-gates was also observed. This latter leakage amounted to about one cubic foot only per second, and the whole turbine gate leakage was found to be 17.4 cubic feet per second with the water surface in the Screen House at Elevation 560.00.

An inspection of the turbine was made before opening the Johnson valve, the manhole cover having been removed from the penstock between the valve and the turbine for the purpose. The Johnson valve was watertight when closed.

Gate Operation. The servomotor indicator used to obtain a record of the gate opening makes a diagram on which the ordinate is time and the abscissa is gate opening on the servomotor scale. The diagram provides an accurate measurement of the gate opening at the time of shut down, and it also records the time during which the load was rejected in each test, thus checking the corresponding time on the Gibson diagram.

Power Measurements.

Three independent readings of the generator output were made, and the average has been accepted as the power output.

Results of Test.

Two tables, included herewith as pages D-79, D-79a and D-80, give the results of the tests. Eight diagrams, included herewith as pages D-81 to D-88 give the same information in graphic form. The diagrams are as follows:

	Title	Page
1.	Power - Discharge Relation (K.W.)	D-81
2.	Power - Discharge Relation (H.P.)	D-82
3.	Hydraulic Efficiency	D-83
4.	Power - Gate Relation	D-84
5.	Discharge - Gate Relation	D-85
6.	Hydraulic Efficiency - Gate Relation	D-86
7.	Hydraulic Efficiency - Horse-power Relation	D-87
8.	First Derivative of Power - Discharge Relation ..	D-88

In all cases the results have been reduced to a uniform head of 305 feet. The first seven of the diagrams are those regularly made in accordance with standard practice for tests of large turbines. The eighth diagram, included as page D-88, entitled "First Derivative of Power - Discharge Relation" is the graphic representation of an original idea evolved by the engineers of the Hydro-Electric Power Commission, and has not hitherto been applied to the test records of hydraulic turbines. It is of great value in determining the proper gate design of any particular turbine as it shows the gain in power for each cubic foot of water added to the flow. It will be noted that for all increments of flow through Turbine No. 5, between the rates of 200 and 1,200 cubic feet per second the gain in power is 34.6 horse-power for each additional cubic foot of water per second. Beyond the rate of 1,200 cubic feet per second,- that is, beyond the point marked "A" on the diagram,- this rate of gain in power decreases in

QUEENSTON-CHIPPAWA POWER DEVELOPMENT
Tests of Unit No. 5
Computations for Turbine Output, Turbine Discharge, Turbine Efficiency
(based on Turbine Tests, May 20th, 1923,
and Generator Tests, July 16th and 21st, 1923)

Test Run Number	Average Generator Output K.W.	Total Generator Losses K.W.	Average Turbine Output K.W.	Equivalent Turbine Output at 305 Feet Head K.W.	Equivalent Turbine Output at 305 Feet Head H.P.
1	48,130	844	48,974	47,860	64,150
2	47,833	837	48,670	47,350	63,450
3	45,170	813	45,983	44,640	59,830
4	41,318	761	42,079	40,840	54,750
5	37,600	717	38,317	37,100	49,730
6	32,133	665	32,798	31,690	42,460
7	26,280	617	26,897	25,890	34,700
8	17,993	566	18,559	17,830	23,900
9	28,830	635	29,465	28,410	38,090
10	33,017	669	33,686	32,470	43,510
11	37,333	715	38,048	36,760	49,260
12	38,767	734	39,501	38,180	51,150
13	38,970	732	39,702	38,460	51,550
14	33,580	683	34,263	33,090	44,340
15	36,080	707	36,787	35,550	47,650
16	44,930	794	45,724	44,230	59,280
17	47,380	833	48,213	46,720	62,600
18	20,500	575	21,075	20,260	27,150

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COPY FOR ENCLOSURE TO Mr. J. Allan Ross. (D-79-a)

QUEENSTON-CHIPPAWA POWER DEVELOPMENT

Tests of Unit No. 5

Computations for Turbine Output, Turbine Discharge, Turbine Efficiency
(based on Turbine Tests, May 20th, 1923,
and Generator Tests, July 16th and 21st, 1923)

Test Run Number	Equivalent Turbine Rate Discharge 305 Feet Head C.F.S.	Turbine Efficiency 305 Feet Head Per Cent.	Correction for Velocity Head at Draft Tube Outlet Per Cent.	Corrected Turbine Efficiency Per Cent.	Gate Opening Per Cent.
1	2,118	.876	.001	.877	1.000
2	2,056	.893	.001	.894	.911
3	1,902	.910	.001	.911	.810
4	1,724	.918	.001	.919	.704
5	1,552	.926	.001	.927	.616
6	1,340	.917	.001	.918	.509
7	1,066	.942	-	.942	.412
8	828	.835	-	.835	.310
9	1,197	.920	-	.920	.463
10	1,354	.929	.001	.930	.524
11	1,529	.931	.001	.932	.606
12	1,609	.920	.001	.922	.644
13	1,606	.928	.001	.929	.641
14	1,380	.929	.001	.930	.539
15	1,463	.942	.001	.943	.583
16	1,838	.932	.001	.933	.784
17	2,007	.902	.001	.903	.875
18	895	.877	-	.877	.340

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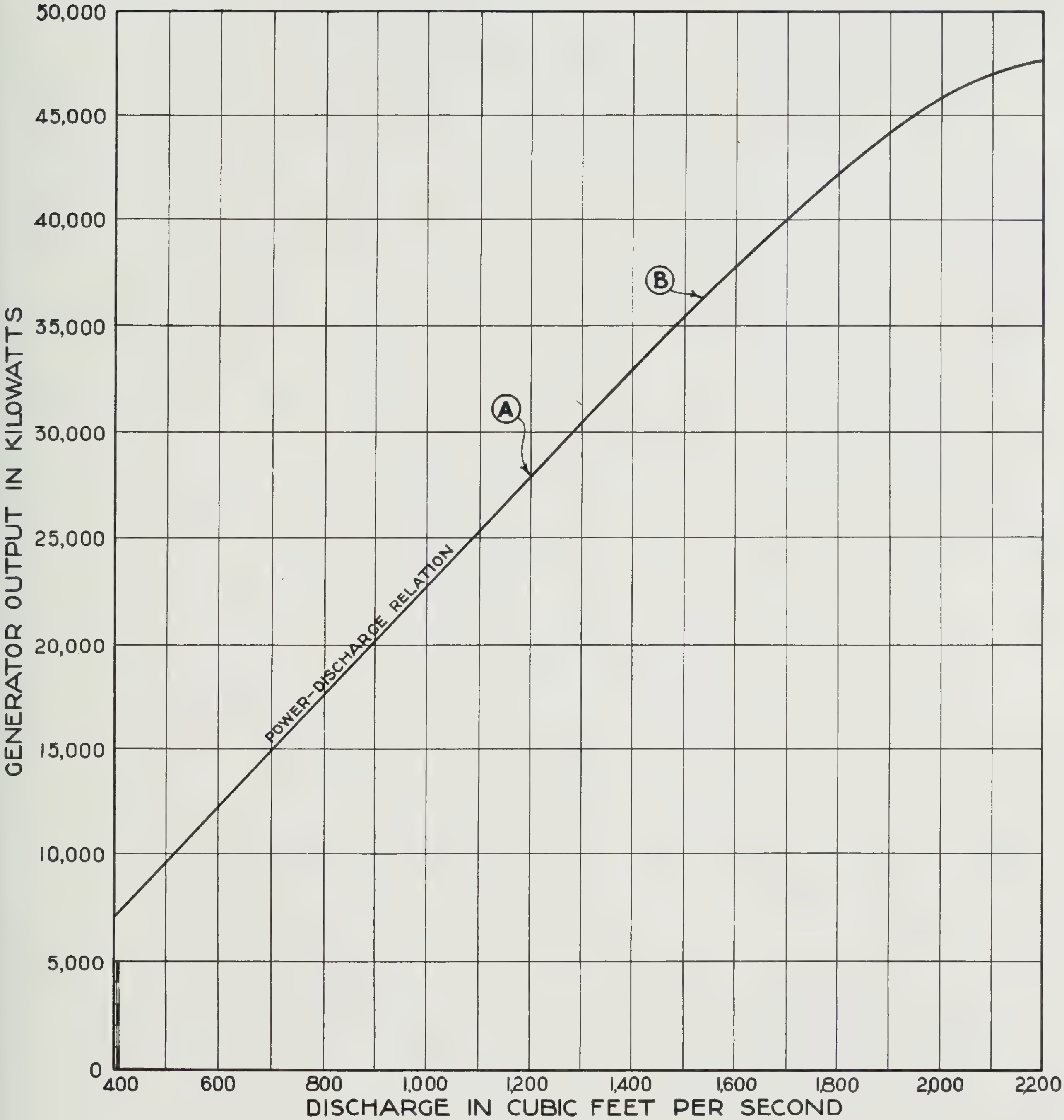
QUEENSTON-CHIPPAWA POWER DEVELOPMENT

Tests of Unit No. 5

Table of Corrected Turbine Output, Discharge and Efficiency for each part of Gate Opening.

Gate Opening — Per Cent.	Turbine Discharge — Cubic Feet Per Second	Turbine Output — Horse-power	Turbine Efficiency — Per Cent.	Correction for Velocity-head at Draft Tube Outlet — Per Cent.	Corrected Turbine Efficiency — Per Cent.
0.00	17.4	0	0	0	0
.05	150.0	-	-	-	-
.10	280.0	[illegible],150	.635	-	.635
.15	408.0	10,700	.759	-	.759
.20	535.0	15,050	.813	-	.813
.30	788.0	23,750	.872	-	.872
.40	1,042.0	32,600	.905	-	.905
.50	1,294.0	41,300	.923	.001	.924
.60	1,519.0	48,860	.930	.001	.931
.70	1,712.0	54,900	.928	.001	.929
.80	1,887.0	59,900	.918	.001	.919
.90	2,043.0	63,230	.895	.001	.896
1.00	2,118.0	64,150	.876	.001	.877

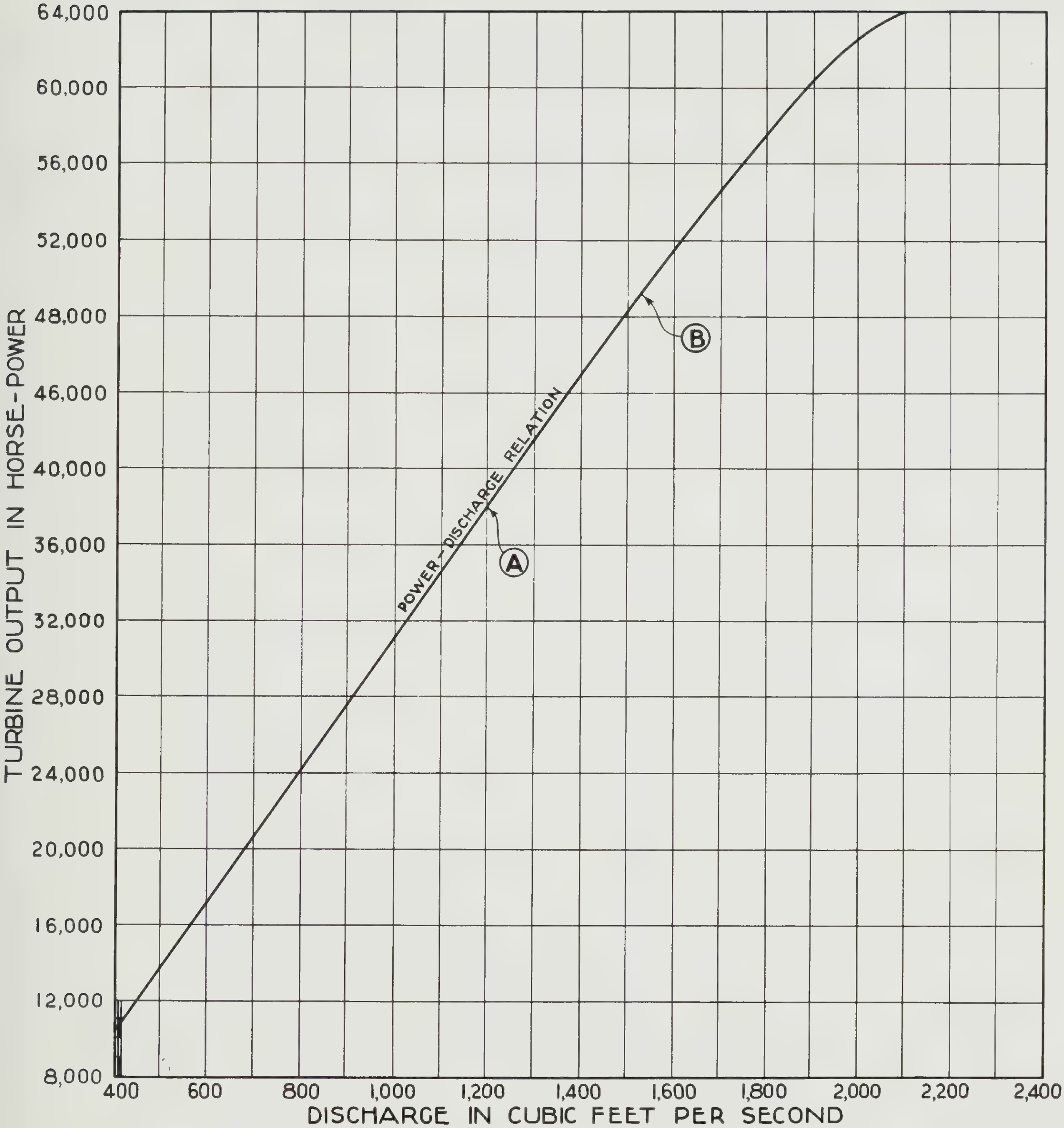


1

HEAD = 305 FEET

SEE TEXT

HYDRO-ELECTRIC INQUIRY COMMISSION
W. D. GREGORY, CHAIRMAN
QUEENSTON-CHIPPAWA POWER DEVELOPMENT
TEST OF TURBINE Nº 5, MAY 20TH., 1923
POWER-DISCHARGE RELATION
Toronto, July 30th., 1923. Made by E.K.D., Checked by W.D.A.
WALTER J. FRANCIS & COMPANY
CONSULTING ENGINEERS



2

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HYDRO-ELECTRIC INQUIRY COMMISSION
W. D. GREGORY, CHAIRMAN
QUEENSTON-CHIPPAWA POWER DEVELOPMENT
TEST OF TURBINE Nº 5, MAY 20TH., 1923
POWER-DISCHARGE RELATION
Toronto, July 30th, 1923. Made by E.H.D., Checked by W.D.R.
WALTER J. FRANCIS & COMPANY
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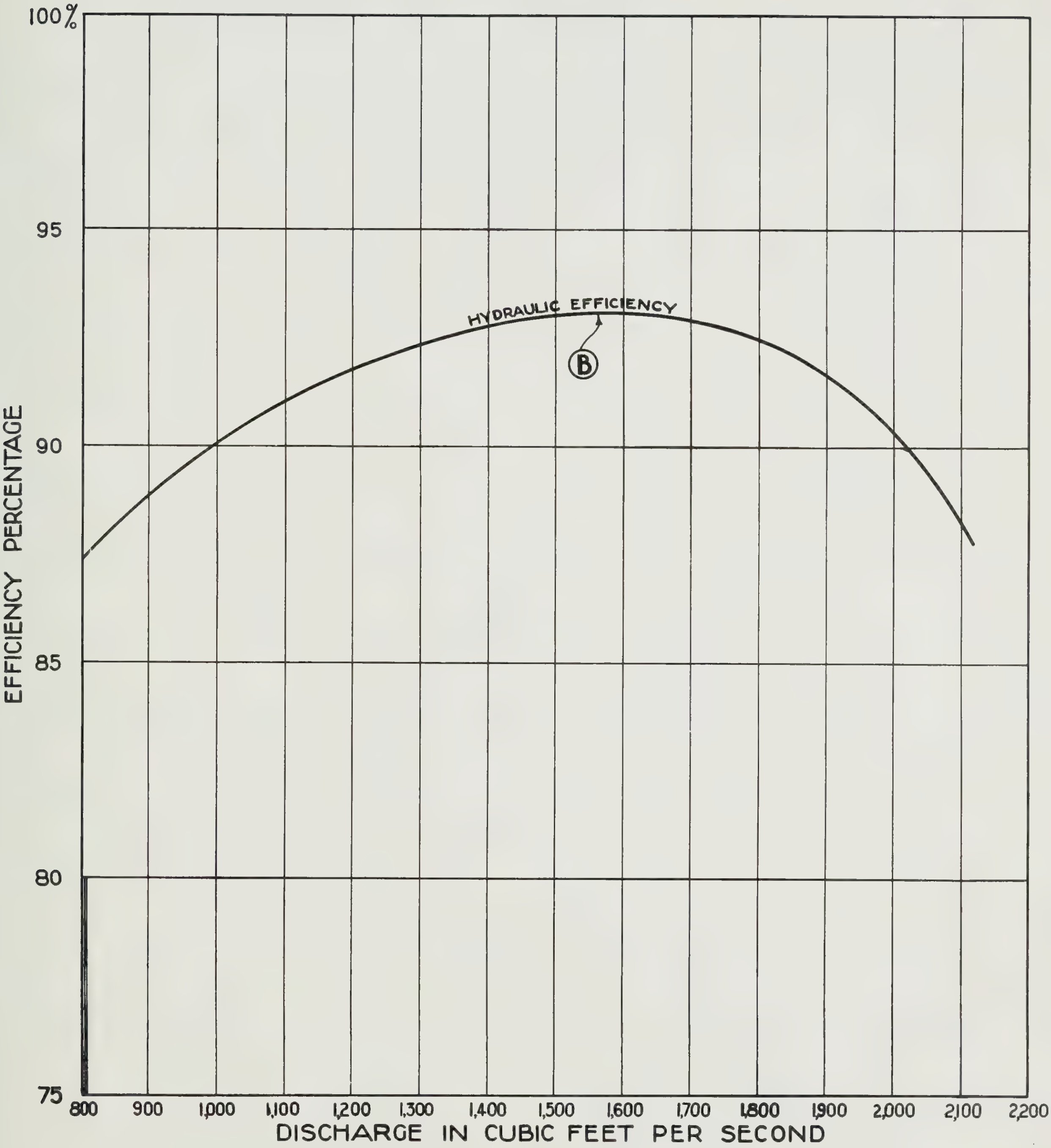

3

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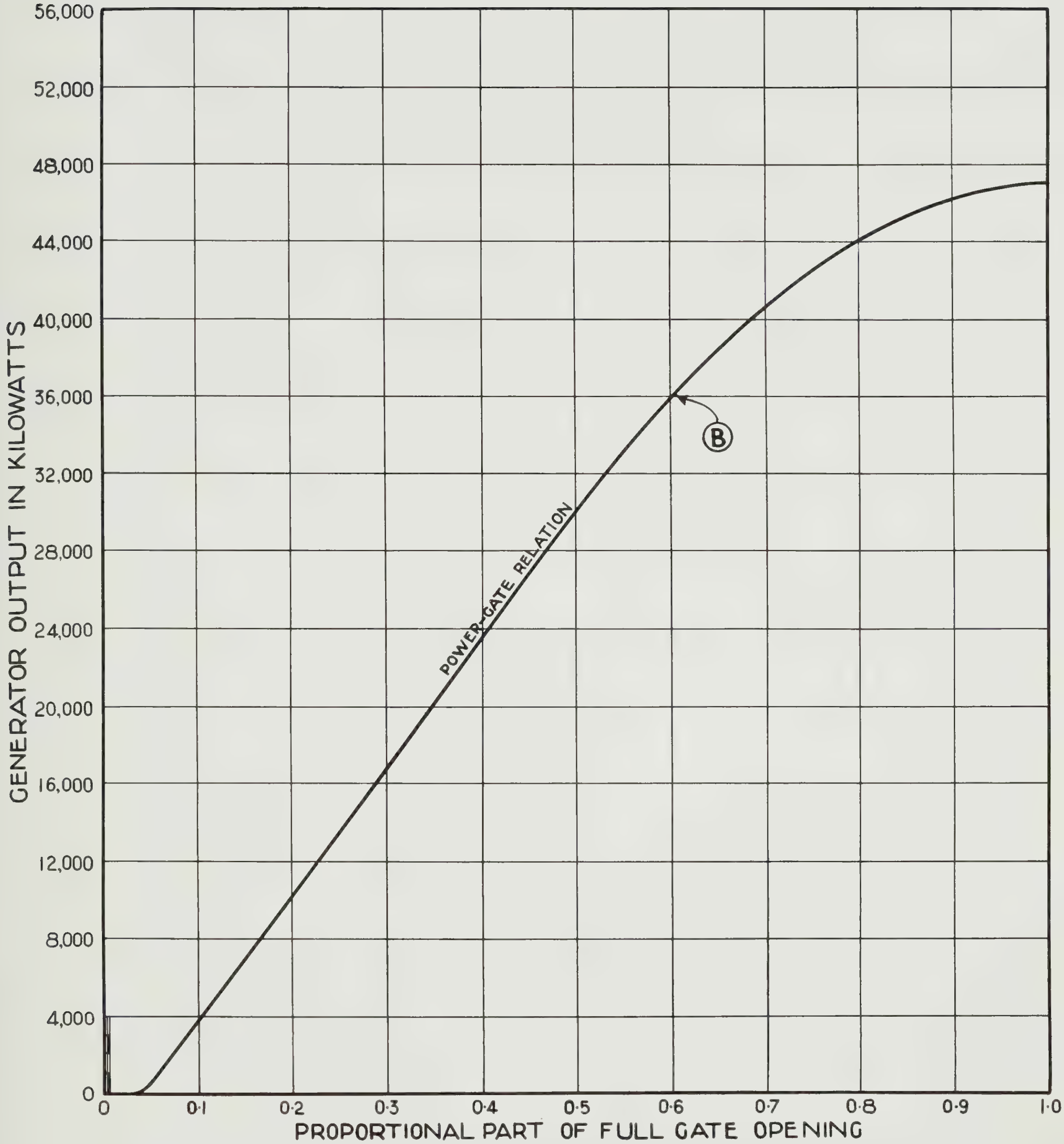
HYDRO-ELECTRIC INQUIRY COMMISSION
W. D. GREGORY, CHAIRMAN
QUEENSTON-CHIPPAWA POWER DEVELOPMENT
TEST OF TURBINE Nº 5, MAY 20TH., 1923
HYDRAULIC EFFICIENCY OF UNIT
Toronto, July 30th., 1923. Made by W.W. Checked by J.W.B.
WALTER J. FRANCIS & COMPANY
CONSULTING ENGINEERS



4

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HYDRO-ELECTRIC INQUIRY COMMISSION
W. D. GREGORY, CHAIRMAN
QUEENSTON-CHIPPAWA POWER DEVELOPMENT
TEST OF TURBINE Nº 5, MAY 20TH., 1923

POWER-GATE RELATION

Toronto, July 30th., 1923. Made by [illegible] Checked by [illegible]
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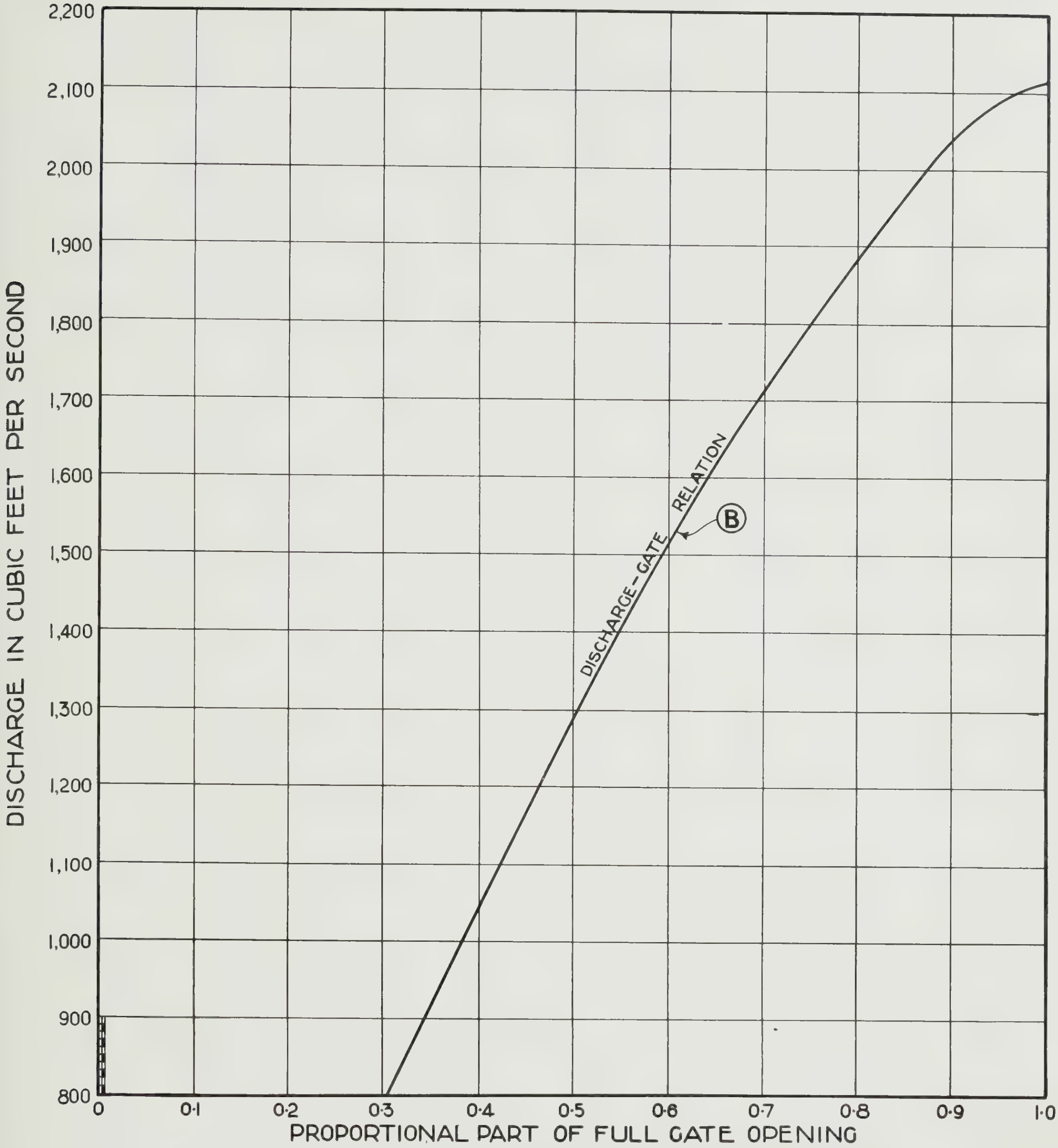

5

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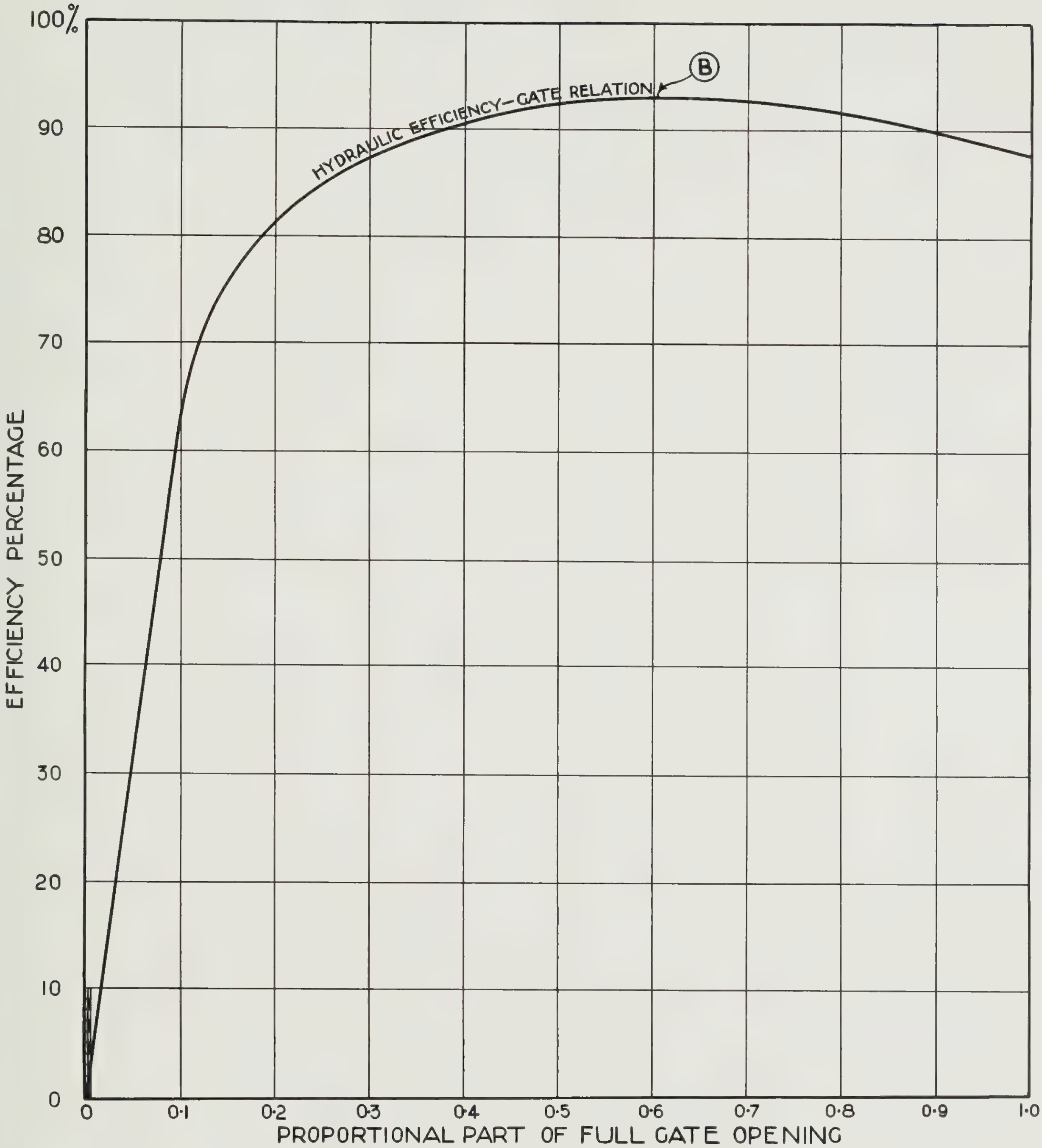
HYDRO-ELECTRIC INQUIRY COMMISSION
W. D. GREGORY, CHAIRMAN
QUEENSTON-CHIPPAWA POWER DEVELOPMENT
TEST OF TURBINE Nº 5, MAY 20TH., 1923
DISCHARGE-GATE RELATION
Toronto, July 30th., 1923. Made by Checked by
WALTER J. FRANCIS & COMPANY
CONSULTING ENGINEERS



6

HEAD = 305 FEET

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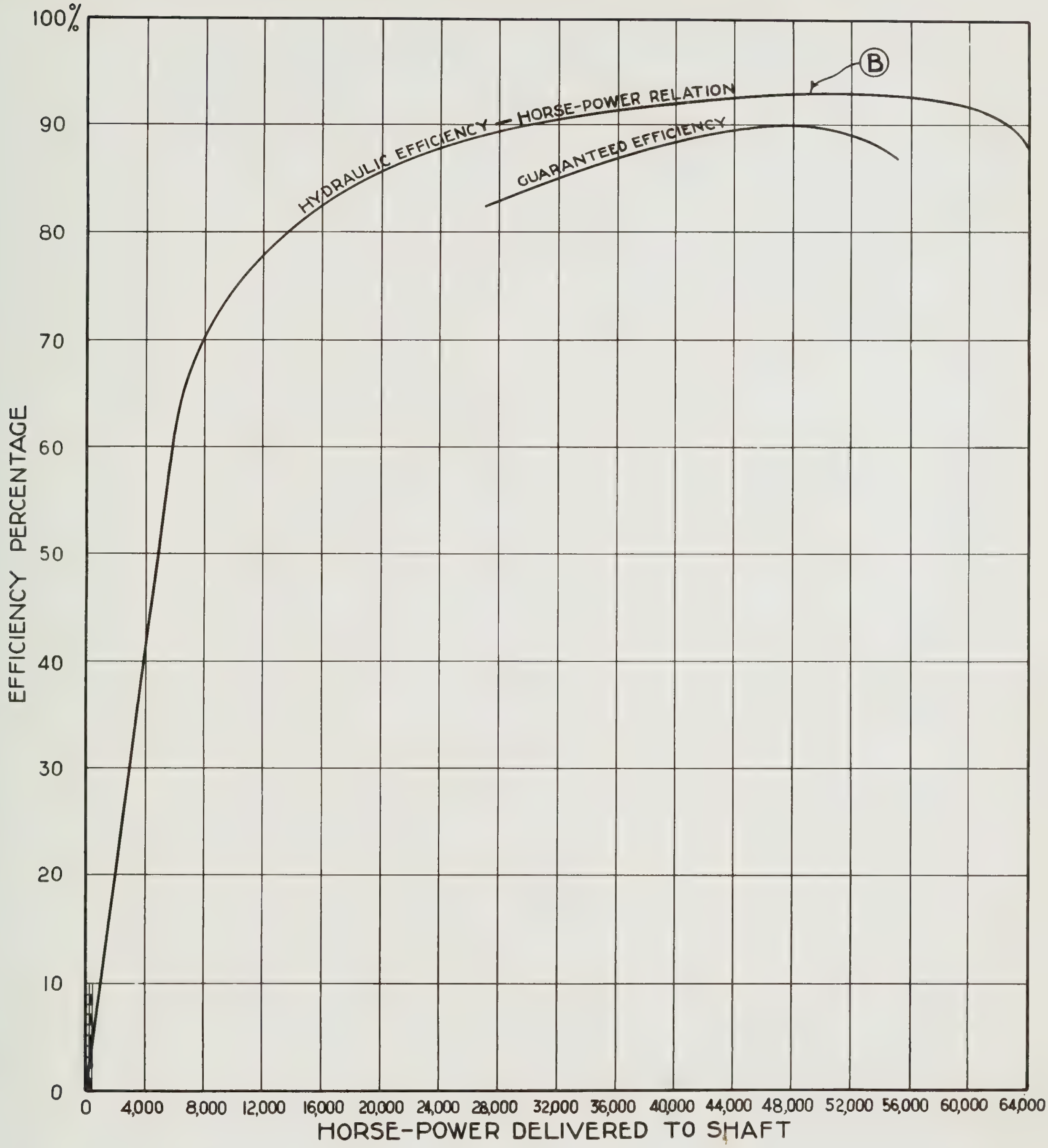
HYDRO-ELECTRIC INQUIRY COMMISSION
W. D. GREGORY, CHAIRMAN
QUEENSTON-CHIPPAWA POWER DEVELOPMENT
TEST OF TURBINE Nº 5, MAY 20TH., 1923
HYDRAULIC EFFICIENCY—
GATE RELATION
Toronto, July 30th., 1923. Made by [illegible], Checked by [illegible]
WALTER J. FRANCIS & COMPANY
CONSULTING ENGINEERS



7

HEAD = 305 FEET

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HYDRO-ELECTRIC INQUIRY COMMISSION
W. D. GREGORY, CHAIRMAN
QUEENSTON-CHIPPAWA POWER DEVELOPMENT
TEST OF TURBINE Nº 5, MAY 20TH., 1923
HYDRAULIC EFFICIENCY —
HORSE-POWER RELATION
Toronto, July 30th., 1923. Made by [illegible], Checked by [illegible]
WALTER J. FRANCIS & COMPANY
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8

HEAD = 305 FEET

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HYDRO-ELECTRIC INQUIRY COMMISSION

W. D. GREGORY, CHAIRMAN

QUEENSTON-CHIPPAWA POWER DEVELOPMENT

TEST OF TURBINE Nº 5, MAY 20TH. 1923

FIRST DERIVATIVE OF POWER–DISCHARGE RELATION

Toronto, July 30th., 1923. Made by W.D., Checked by W.D.

WALTER J. FRANCIS & COMPANY

CONSULTING ENGINEERS

the manner shown by the curve.

The point marked "B" on the various curves is that of the maximum efficiency of the unit, 93.1 per cent., which occurs with a discharge of 1,530 cubic feet per second, equivalent to a power output of about 49,300 horsepower. It will be noted that between four-tenths of full gate opening and nine-tenths of full gate opening the efficiency of the turbine is over 90 per cent.

Tests of Generator No. 5.

Between July 17th and 21st, 1922, a series of tests was made on Generator No. 5 and its exciter in the Queenston-Chippawa plant to determine the efficiency and the other characteristics of the unit. The tests of the generator and exciter were witnessed by Mr. Francis, by Mr. J. Kynoch, Chief Engineer, Canadian General Electric Company, and by Mr. B. L. Barns, B.Sc., Designing Engineer, Canadian General Electric Company. Mr. Acres was also present during part of the tests, which were under the charge of G. D. Floyd and D. B. Fleming, Assistant Engineers of the Electrical Engineering and Laboratories Department of the Hydro-Electric Power Commission of Ontario. A staff of qualified engineers was used for recording and calculating the various phenomena and results.

Tests for the measurement of core loss, windage and friction were made by disconnecting Generator No. 5 from its turbine and driving it as a synchronous motor by means of Unit No. 4, and the results noted by means of the input method and the deceleration method.

Tests were made to determine the efficiency of the generator, both exclusive

and inclusive of the exciter losses, the losses in the exciter being obtained from the records of the tests at the factory. Determinations of the losses in armature copper and field copper were also made by the resistance method, and the stray losses computed. Calculations were made for various power factors and percentages of load for two different temperature conditions. The open circuit saturation curve, the short circuit impedance for three phases, for single-phase between terminals, and for single-phase to neutral, the zero power factor saturation curve for half load and for full load, and the regulation of the machine, were also determined.

A heat run with the generator operating at about 36,000 kilowatts and at its rated power factor was made, during which the quantity of air required for cooling was also measured. A heat run was also taken for the loaded exciter unit.

The above tests on Generator No. 5 are all that have been practicable to carry out up to the present time, and they include all tests necessary in relation to the test of Turbine No. 5, in conjunction with which they have been used. Analysis of the results obtained shows that the unit meets the guarantees of the makers on all points covered by the tests so far made. The tests remaining to be made are those for overspeed, sudden short circuit, oscillograms and analysis of wave form, and a high potential test at 80,000 volts.

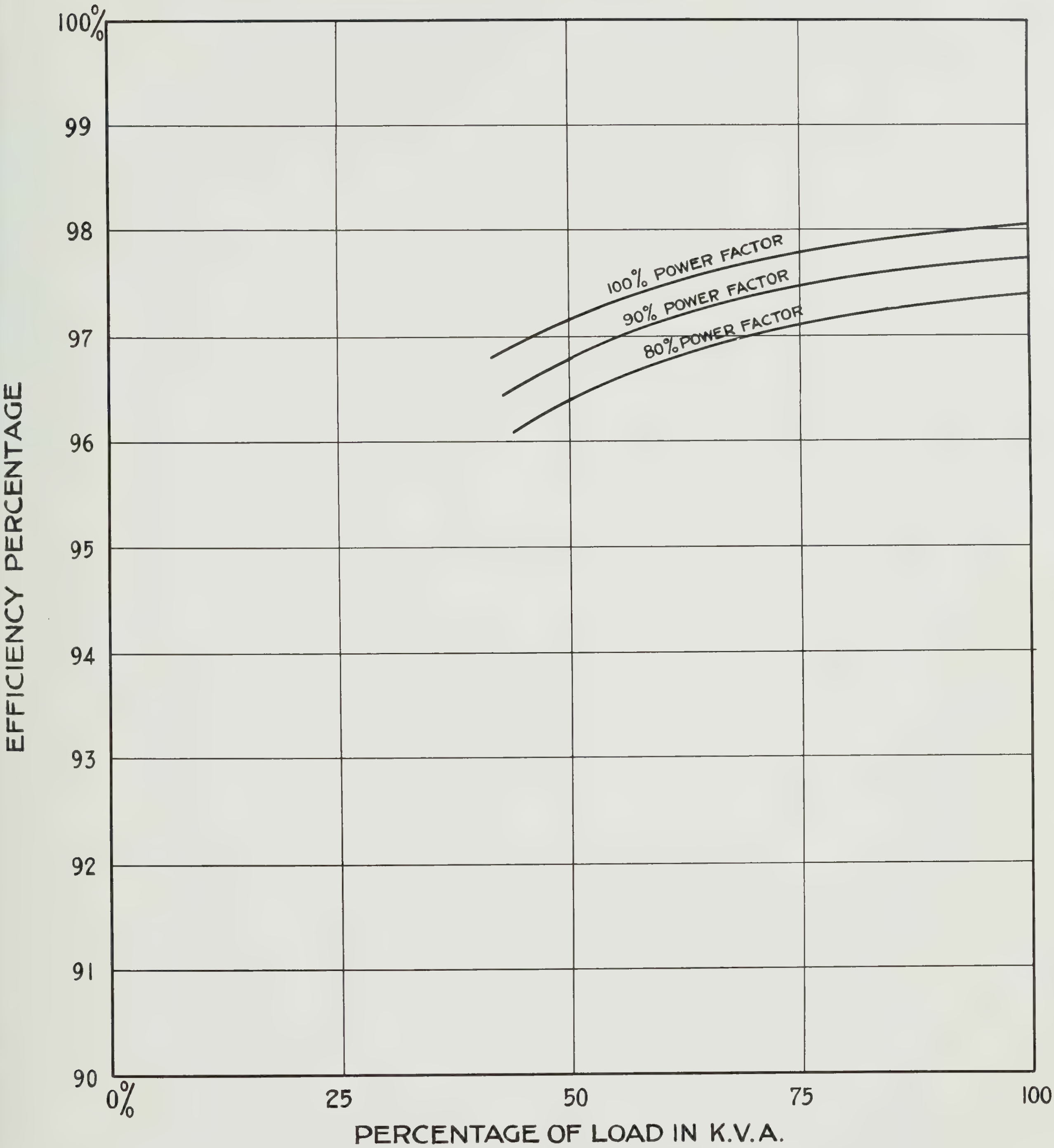
As the efficiency of the generator is the principal electrical item affecting the power output, we have prepared a diagram, included as page D-91, on which are plotted curves representing the efficiency of Generator No. 5 at 90 degrees Centigrade temperature of the stator windings and including the exciter

and [illegible] of the [illegible]. The [illegible] in the [illegible] from the [illegible] as the [illegible] of the [illegible] generator output and field [illegible] the [illegible] and the [illegible]. Calculations [illegible] and percentages of load [illegible] the different temperature [illegible]. The [illegible] saturation curve, the short circuit [illegible] for three phase, for single-phase between terminals, and for single-phase to neutral, the zero power factor saturation curve for half load and for full load, and the regulation of the machine, were also determined.

A heat run with the generator operating at about [illegible] kilowatts [illegible] its rated power factor [illegible] the quantity of air [illegible] the building was also measured. A [illegible] for the [illegible] unit.

The above tests on Generator No. 5 are all that have been practicable to carry out up to the present time, and they include all tests necessary in relation to the test of Turbine No. 5, in conjunction with which they have been used. Analysis of the results obtained above that the unit meets the guarantees of the makers on all points covered by the tests so far made. The tests remaining to be made are those for [illegible], [illegible], and analysis of wave form, and a high potential test at [illegible] volts.

As the efficiency of the generator is the principal [illegible] the power output, we have prepared a diagram, included on page [illegible], on which are plotted curves representing the efficiency of Generator No. 5 at [illegible] degrees Centigrade temperature of the stator windings and including the [illegible]



HYDRO-ELECTRIC INQUIRY COMMISSION

W. D. GREGORY, CHAIRMAN

QUEENSTON-CHIPPAWA POWER DEVELOPMENT

TEST OF TURBINE No. 5, MAY 20TH., 1923

EFFICIENCY OF GENERATOR

Toronto, July 30th, 1923. Made by G.B.B. Checked by W.D.A.

WALTER J. FRANCIS & COMPANY

CONSULTING ENGINEERS

losses, for power factors of 80 per cent., 90 per cent. and 100 per cent. The base line of the diagram represents the percentage of load in kilovolt-amperes. It will be noted that the efficiency of the generator at full load and 100 per cent. power factor is slightly over 98 per cent., while at 80 per cent. power factor the full load efficiency is about 97.4 per cent.

Judging by the operation of the other four generators in the Power House, and by such tests as have been made upon them, it would appear that the characteristics of all five units are quite similar and that there is little practical difference amongst them. The tests of Generator No. 5 may therefore be taken as indicative of the average to be expected for the full development.

By combining the efficiency curves of the generator and the exciter with those of Turbine No. 5, the overall efficiency of Unit No. 5 can be obtained for any percentage of output and for any power factor.

Overall Efficiency of the Power Plant.

The results show that over a wide range of output the Queenston-Chippawa power plant may be operated so as to give over 90 per cent. efficiency from the Forebay to the Tailrace, including all hydraulic, mechanical and electrical losses up to the 12,000-volt bus bars in the generating station. Based on this data and all other data available, it would appear that 550,000 electrical horsepower delivered to the 12,000-volt bus bars may be considered as the commercial capacity of the Queenston-Chippawa Power Development.

Walter J. Francis
Consulting Engineer.

Toronto, July 30th, 1923.

Curves, for power factors of 80 per cent., 90 per cent. and 100 per cent. The base line of the diagram represents the percentage of load in kilovolt-amperes. It will be noted that the efficiency of the generator at full load and 100 per cent. power factor is slightly over 98 per cent., while at 80 per cent. power factor the full load efficiency is about 97.4 per cent.

Judging by the operation of the other four generators in the Power House, and by such tests as have been made upon them, it would appear that the characteristics of all five units are quite similar and that there is little practical difference amongst them. The tests of Generator No. 2 may therefore be taken as indicative of the average to be expected for the full development.

COPY

By combining the efficiency curves of the generator and the [illegible] [illegible] those of Turbine No. 2, the overall efficiency of Unit No. 2 can be obtained for any percentage of output and for any power factor.

Overall Efficiency of the Power Plant.

The results show that over a wide range of output the Queenston-Chippawa power plant may be operated so as to give over 90 per cent. efficiency from the forebay to the tailrace, including all hydraulic, mechanical and electrical losses up to the 12,000-volt bus bars in the generating station. Based on this data and all other data available, it would appear that 500,000 electrical horsepower delivered to the 12,000-volt bus bars may be considered as the commercial capacity of the Queenston-Chippawa Power Development.

Walter J. Francis
Consulting Engineer.

Toronto, [illegible], 1922.